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PHASE I INSPECTION REPORT

NATIONAL DAM SAFETY PROGRAM

RAINY CREEK BASIN
ZONOLITE TAILINGS DAM

LIBBY, MONTANA
MT-1470

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CONSTRUCTION PRODUCTS DIVISION
(OWNER AND OPERATOR)

PREPARED BY:

MORRISON - MAIERLE, INC.
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EXECUTIVE SUMMARY

Under contract with the State of Montana Department of Natural Resources and Conservation and with representation from Department of Natural Resources and Conservation and the Construction Products Division of W.R. Grace and Company, Morrison-Maierle, Inc. inspected Zonolite Tailings Dam on 25 July 1980 under the authority of Public Law 92-367. At that time the project was undergoing Phase 5 construction to raise the dam 35 feet. A followup inspection was conducted on 13 August 1981 upon construction completion. The dam is located in Lincoln County about 6.5 miles northeast of Libby, Montana in the Rainy Creek Basin.

This report was compiled from information obtained during onsite inspections, review of the construction log and plans, and analysis of available information. Findings were compared with engineering criteria that are currently accepted by most private and public agencies engaged in dam design, construction and operation.

FINDINGS AND EVALUATION

Zonolite Tailings Dam is owned and operated by W.R. Grace and Company and is located on private land. The reservoir is used to contain mine tailings. As tailings are deposited, the dam is raised in phases to provide storage. The dam, designed by Bovay Engineers, Inc. of Spokane, Washington, and Harding-Lawson Associates of Novato, California, will eventually be 200-feet high. Presently, it is 135-feet high (crest elevation 2,925 feet NGVD). The reservoir normally stores only enough water to accomplish tailings settling. A 48-inch diameter corrugated metal pipe intercepts Rainy Creek above the 3,000 foot elevation and diverts flow downstream of the dam. Existing manual controls at the diversion structure do not prevent storm flows from entering the reservoir. This report evaluates the 135-foot high dam assuming a tailings elevation of 2,873.0 feet NGVD (July 1980 inspection) and a dam crest elevation of 2,925 feet NGVD. Under these conditions, the project is capable of impounding about 2,120 acre-feet of water at spillway crest elevation 2,920 feet NGVD and 2,450 acre-feet at dam crest elevation 2,925 feet NGVD. All elevations used in this report are based on owner-supplied design drawings which correspond to the National Geodetic Vertical Datum (NGVD).

On the basis of criteria in U.S. Army Corps of Engineers Recommended Guidelines for Safety Inspection of Dams (Reference 1), the project is large in size. The dam is located such that its failure could endanger more than a few lives and cause excessive economic loss. However, no dam breach analysis or routing of a dam breach flood was made for the downstream area. The conclusions on probable damage are based on brief field visits and engineering judgement.

The project is classified as having a high (Category 1) downstream hazard potential. Inspection criteria (Reference 1) recommend that a large size project with a high downstream hazard potential be capable of safely handling the probable maximum flood (PMF). The PMF is the flood expected from the most severe combination of meteorologic and hydrologic conditions that are reasonably possible in the region.

An estimated thunderstorm PMF was developed for the 9.7-square mile drainage basin during this dam safety study. The PMF resulting from the 6-hour thunderstorm has an estimated volume of 3770 acre-feet and a peak flow of 43,400 cfs The spillway has a maximum discharge capacity of 200 cfs with the reservoir at assumed top of dam, elevation 2,925 feet. The decant tower with minimum inlet elevation at 2890.1 feet has a maximum discharge capacity of 18 cfs and is used to maintain a steady level on the settling pond so that tailings will settle out and only clear water is discharged downstream. The routing of the PMF was started with the reservoir four feet above the inlet elevation of the decant line, which is the anticipated reservoir elevation prior to the occurrence of a flood of this magnitude. Routing of the PMF indicates that the dam is overtopped when approximately 55 percent of the total flood volume enters the reservoir. A flood with a hydrograph having ordinates corresponding to 45 percent of the PMF hydrograph ordinates is just controlled by the project. Larger floods would overtop the dam. These percentages of the PMF are valid only for conditions immediately after the completion of Phase 5. As tailings are deposited, there will be less storage available for floodwaters and the dam will overtop at PMF percentages smaller than those indicated above. The dam is constructed of materials that could quickly erode and rapidly fail if overtopped by floodwaters. Such failure could endanger lives immediately downstream at the screen plant and cause extensive damage to the highway, property and buildings. Because the project cannot safely handle the recommended spillway design flood (SDF), which is the full PMF, Zonolite Tailings Dam does not conform with inspection guideline hydraulic and hydrologic criteria.

During construction of the various phases of the dam, Bovay Engineering (for the starter dam only) and Harding-Lawson provided engineering services that included observation, consultation, and material testing. Visual inspection of the dam revealed no evidence of cracking, settlement or slope instability. Seepage control measures appeared to be functioning as designed. The structural integrity and support of the spillway is questionable and requires attention to insure flows do not adversely affect embankment safety. There is no riprap or erosion protection on the embankment slopes. Rill erosion is evident but is controlled by seasonal maintenance. Review of the stability analysis on file with the owner indicates the analysis is adequate and that Zonolite Tailings Dam conforms with inspection guideline stability criteria.

RECOMMENDATIONS

Develop and immediately place in action a downstream warning plan for use in the event of possible dam overtopping or structural failure. Periodically test the decant line for possible leaks within the embankment and perform necessary maintenance and repairs. Conduct more detailed hydrologic and hydraulic routing studies to better determine downstream hazard potentia

and to establish the minimum safe flood storage volume and spillway capacity. Studies should take into account the continually decreasing water storage volume as tailings are deposited, the nature of the tailings and their effect of sudden release on the downstream environment and an evaluation of the structural adequacy of the spillway under a full range of possible flow conditions. Modify the operation and/or project as studies indicate. Continue to monitor and evaluate seepage and conduct periodic inspections of the project on at least an annual basis by engineers experienced in dam design and construction.

Prior to performing engineering studies or remedial construction, coordinate with applicable Federal and State agencies to insure compliance with all pertinent laws and regulations.

Rodger C. Foster
Professional Engineer

PERTINENT DATA
ZONOLITE TAILINGS DAM

1. GENERAL

Federal ID No.	MT-1470
Owner	W.R. Grace & Company
Operator	W.R. Grace & Company
Date Constructed	Original 1971 Expanded 1973, 1975, 1977, 1980
Location	Section 22, T31N, R30W Longitude 115°24'40" Latitude 48° 26'32"
County, State	Lincoln County, Montana
Watershed	Rainy Creek
Size	Large
Downstream Hazard Potential	Category 1, High
USGS Quadrangle	Vermiculite Mountain, Montana

2. RESERVOIR (Phase 5)

Surface Area at Spillway Crest	68.5 acres
Drainage Area	9.7 square-miles
*Storage at Spillway Crest (elevation 2920 feet NGVD)	2120 acre-feet
*Storage at Dam Crest (elevation 2925 feet NGVD)	2450 acre-feet
*Storage at Decant Line Inlet (elevation 2890.1 feet NGVD)	550 acre-feet
Surcharge Storage	330 acre-feet
Reservoir Elevation (25 July 80) during construction of Phase 5 (13 August 81)	2880.0 feet, NGVD 2893.8 feet, NGVD

*Water storage only
(Assumed tailings elevation 25 July 80, 2873.0 ft.)

3. SPILLWAY (Phase 5)

Type	Uncontrolled chute
Shape	Half section, 8-foot diameter round corrugated metal pipe.
Crest Elevation	2920 feet, NGVD
Capacity with Reservoir at Dam Crest	200 cfs

4. OUTLET WORKS/DECANT TOWER (Phase 5)

Decant Tower	6-foot diameter steel pipe, set vertically with a 1.7-foot wide entrance extending the full height of pipe. Floor elevation at 2890.1 feet.
Decant Line	1600 feet of 16 inch diameter welded steel pipe at 1% slope.
Gate	Uncontrolled overflow weir to wet well inlet structure.
Capacity with Reservoir at Dam Crest:	18 cfs

5. DAM (Phase 5)

Type	Compacted Tailings
Length	1154 feet at elevation 2925 feet NGVD
Crest Width	40 feet at elevation 2925 feet NGVD
Crest Elevation	2925 feet NGVD
Hydraulic Height crest to toe	135 feet
Upstream Slope	1V on 2H
Downstream Slope	1V on 2H w/Construction Terraces

Chapter 1
BACKGROUND

1.1 INTRODUCTION

1.1.1 Authority and Scope

This report summarizes the Phase I inspection and evaluation of the Zonolite Tailings Dam, owned by W.R. Grace and Company, Construction Products Division.

The National Dam Inspection Act, Public Law 92-367 dated 8 August 1972, authorized the Secretary of the Army, through the Corps of Engineers, to conduct safety inspections of non-Federal dams throughout the United States. Pursuant to that authority, the Chief of Engineers issued "Recommended Guidelines for Safety Inspection of Dams" in Appendix D, Volume 1 of the U.S. Army Corps of Engineers' report to the United States Congress on "National Program of Inspection of Dams" in May 1975.

The recommended guidelines were prepared with the help of engineers and scientists highly experienced in dam safety from many Federal and State agencies, professional engineering organizations and private engineering consulting firms. Consequently, the evaluation criteria presented in the guidelines represent the comprehensive consensus of the engineering community.

Where necessary, the guidelines recommend a two-phased study procedure for investigation and evaluation of existing dam conditions, so deficiencies and hazardous conditions can be readily identified and corrected. The Phase I study is:

(1) a limited investigation to assess the general safety condition of the dam.

(2) based upon an evaluation of the available data and a visual inspection.

(3) performed to determine if any needed emergency measures and/or if additional studies, investigations, and analyses are necessary or warranted.

(4) not intended to include extensive explorations and analyses or to provide detailed alternative correction recommendations.

The Phase II investigation includes all additional studies necessary to evaluate the safety of the dam. Included in Phase II, as required, should be additional visual inspections, measurements, foundation exploration and testing, material testing, hydraulic and hydrologic analyses, and structural stability analyses.

The authority for the Corps of Engineers to participate in the inspection of non-federally-owned dams is limited to Phase I investigations with the exception of situations of extreme emergency. In these cases, the Corps may proceed with Phase II studies but only to the extent needed to answer serious questions relating to dam safety that cannot be answered

otherwise. The two phases of investigation outlined above are intended only to evaluate project safety and do not encompass in scope the engineering required to perform design or corrective modification work. Recommendations contained in this report may be for either Phase II safety analyses or detailed design study for corrective work.

The responsibility for implementation of these Phase I recommendations rests with the dam owner and the State of Montana. It should be noted that nothing contained in the National Dam Inspection Act, and no action or failure to act under this Act shall be construed (1) to create liability in the United States or its officers or employees for the recovery of damage caused by such action or failure to act or (2) to relieve an owner or operator of a dam of the legal duties, obligations, or liabilities incident to the ownership or operation of the dam.

1.1.2 Purpose

The purpose of the inspection and evaluation is to identify conditions that threaten public safety, so that they may be corrected in a timely manner by non-Federal interests.

1.1.3 Inspection

The findings and recommendations in this report are based on brief visual inspections of the project and a detailed review of available construction plans, analyses and reports. Inspection procedures and criteria are those established by the Recommended Guidelines for Safety Inspection of Dams (Reference 1).

Personnel present during the 25 July 1980 inspection included:

Larry Tegg,	State of Montana, Department of Natural Resources and Conservation
Rodger Foster,	Team Leader, Morrison-Maierle, Inc. Water Resource Engineer
Mike Kaczmarek,	Engineering Geologist, Morrison-Maierle, Inc.
Robert Foss,	Chief Engineer for W. R. Grace and Company, Zonolite Operations at Libby.

Those present for the 13 August 1981 inspection were:

Art Taylor,	State of Montana, Department of Natural Resources and Conservation
Harold Eagle,	Chief Engineer, Morrison-Maierle, Inc.
Mike Kaczmarek,	Engineering Geologist, Morrison-Maierle, Inc.
Phil Porrini,	Water Resources Engineer, Morrison-Maierle, Inc.
Michael Ray,	Chief Engineer for W.R. Grace and Company, Zonolite Operations

Additional Morrison-Maierle personnel who contributed to the evaluation are:

Bill Keith, Structural Engineer

Ken Salo, Hydrologist/Hydraulics Engineer.

Subsequent discussions and coordination were conducted with Mr. Lyle Lewis of Harding-Lawson Associates and Messers William McCaig and Michael Ray of W.R. Grace Company, concerning the completed Phase 5 addition to the dam.

This report was reviewed by W.R. Grace and Company-Construction Products Division, the Montana Department of Natural Resources and Conservation, the Mine Safety and Health Administration and Mr. Lyle Lewis of Harding-Lawson Associates. Mr. Lewis submitted verbal comments only. The written comments received are included in the appendix.

1.2 DESCRIPTION OF PROJECT

1.2.1 General

Zonolite Tailings Dam and Reservoir are located at the site of the W.R. Grace and Company vermiculite mine on Rainy Creek about 6.5 miles northeast of Libby, Montana (Plates 1 and 2) (Photos 1 and 2). Rainy Creek empties into the Kootenai River 5.5 miles upstream of Libby. The project's Federal identification number is MT-1470. The 135-foot high tailings dam creates a reservoir that normally stores only enough water to allow mine tailings to settle out (about 7 feet deep). Normal flow from the Rainy Creek drainage basin is diverted around the reservoir through a 48-inch diameter corrugated metal pipe (CMP) (Photo 5). With the tailings at assumed elevation 2,873.0 feet and the dam crest at elevation 2,925 feet, the dam is capable of impounding about 2,450 acre-feet of water to the dam crest. As tailings accumulate, storage volume decreases. In time, the dam is raised to provide needed storage. The dam is planned to eventually be 200-feet-high. Based on visual reconnaissance and engineering judgement, the screening plant and product storage area at the mouth of Rainy Creek and State Highway 37 could be affected by a sudden breach of the tailings dam. In accordance with the Recommended Guidelines, the project is large in size and the downstream hazard potential is high (Category 1).

Outlets from the reservoir consist of an uncontrolled chute spillway in the left abutment and a decant structure located in the upper reservoir area (Photos 3 and 6). The spillway consists of a trapezoidal approach channel which transitions to a half-section of an 8-foot diameter corrugated metal pipe. The decant structure is a weir-controlled inlet structure to a 16-inch diameter welded steel pipe which extends along the west shoreline in the reservoir, through the dam embankment and discharges into the natural drainage downstream of the dam. The decant tower is designed to skim the cleaner surface water from the tailings pond.

1.2.2 Regional Geology and Seismicity

The Zonolite Tailings Dam is located in the northwest corner of Montana in the Rocky Mountain physiographic province (Reference 2). The area is characterized by high, rugged north-northwest-trending mountain ranges separated by narrow linear valleys that parallel the ranges. The mountain ranges are composed of late Precambrian Belt Series strata consisting of fine-grained clastic and carbonate rocks.

The Belt Series rocks range from 17,000 to 40,000 feet thick (Reference 3) and have undergone regional low-grade metamorphism. The Belt Series strata near Libby and the Zonolite Tailings Dam are deformed into broad, open, north-northwest-trending folds. High angle normal faults of regional proportions parallel the trend of the folds. The narrow linear valleys of the region are fault-bounded structural troughs. Surficial deposits are present on the floor of the Kootenai River valley and its major tributaries and consist primarily of alluvial sand and gravel, glacial lake silts, and alpine glacial deposits.

The Zonolite Tailings Dam is located on an elongate intrusive rock body that intrudes strata of the Precambrian Wallace Formation within the trough of a north-west-trending synclinal fold. The intrusive body is named the Rainy Creek stock and is a complex, composite intrusive which encompasses about 7.5 square miles of outcrop area underlying Vermiculite Mountain and most of the Rainy Creek valley west of Vermiculite Mountain. Vermiculite is mined from this intrusive.

In accordance with the Guidelines' Seismic Zone Map (Reference 1), the Zonolite Tailings Dam is in Seismic Zone 2. The seismic probability of Zone 2 has a potential for moderate earthquake damage and is based on known distribution of damaging earthquakes. Stability analysis performed for the planned 200 foot high embankment indicate that the computed safety factors exceed minimum recommended allowable safety factors for static conditions and also for seismic loadings of 0.1 and 0.2 times gravitational acceleration (Reference 6). Recommended Guidelines indicate that no hazard to embankment dams from earthquakes generally exists in Seismic Zone 2 provided static conditions are satisfactory and conventional safety margins exist.

1.2.3 Site Geology

The geology along the foundation of the Zonolite Tailings Dam was explored by means of 8 test borings 29 to 55 feet deep, 14 test pits 7 to 20 feet deep, 3 seismic velocity survey lines, and 27 soils resistivity soundings all conducted under the supervision of Harding-Lawson Associates in December, 1970 and May, 1971. The seismic velocity lines were used to determine depths to bedrock below the unconsolidated surficial deposits in the foundation area. The resistivity soundings were used to confirm the continuity of unconsolidated deposits between test boring locations. Field techniques and general methodology used are described in Appendix B of Lewis and Lawson (Reference 6).

A summary description of the preconstruction site geology is provided in Appendix A of Lewis and Lawson (Reference 6). Field observations conducted at the time of the dam safety inspection in the area of the abutments

and foundation were in agreement with the 1971 geologic report. The July 25, 1980, field inspection revealed bedrock underlying the dam site in the left abutment consists of the magnetite pyroxenite described by Boettcher (1963). The bedrock in the foundation area and in the right abutment area was covered by alluvium and glacial deposits and was not observed during the inspection. The bedrock exposed on the right abutment above the elevation of the existing dam (elevation 2890 feet) appeared to be the least altered, finer-grained pyroxenite of Boettcher (1963).

The magnetite pyroxenite observed in construction cuts on the downstream side of the left abutment on July 25, 1980, was a highly weathered, friable rock. As described in Appendix A of Lewis and Lawson (Reference 6), the "upper few feet of pyroxenite bedrock has physical characteristics more like those of a dense sand than rock." Discontinuous shear planes 2 to 3 inches thick oriented parallel to the valley wall were observed in the left abutment rocks on about 2 to 3 foot spacing. Lewis and Lawson (Reference 6) interpret the thin shear zones, which contain silt and clay sized rock gouge, to be the result of either glacial ice loading and/or gravity induced rock creep. The left abutment bedrock is discontinuously covered with as much as 10 feet of weathered rock debris. Lewis and Lawson (Reference 6) describe about 4 feet of highly permeable clean sand and gravel present as an outwash terrace remnant at about elevation 2830 feet.

Test pits and borings show the unconsolidated surficial deposits in the right abutment area consist of glacial outwash and till as much as 40 feet thick up to about elevation 2,890 feet (Reference 6). Cuts on the right abutment near elevation 2,870 feet on July 25, 1980, exposed fluvially bedded gravelly coarse sand (outwash). Subsurface investigations (Reference 6) of the unconsolidated foundation materials show as much as six feet of soft silt with lenses of fine sand and sparse gravel stringers over coarse gravel outwash with lenses of silt and fine sand and zones of quartzite boulders 4 to 5 feet in size. The depth to bedrock below the land surface on the alluvial silts and the gravel outwash ranges from 26 to 45 feet in the test boring logs (Reference 6).

1.2.4 Design and Construction History

The Zonolite Tailings Dam was designed in 1971 for W. R. Grace & Company by Bovay Engineers, Inc. of Spokane, Washington and Harding-Lawson Associates of Novato, California. The dam was designed as a tailings impoundment dam to retain fine tails produced in the vermiculite milling process. The staged construction plan called for the construction of a starter dam with provisions for raising the dam in stages as the storage capacity was depleted by tailings. The starter dam was constructed immediately downstream of an older existing dam. This older dam is identified in Reference 6 by crest elevation (2830 feet) and centerline location.

The 50-foot high starter dam was completed in November 1971 to an elevation of 2850 feet. Since that time three additional phases have been completed. Under the phase 1 expansion, completed in June 1973, the dam was raised to elevation 2875 feet. Phase 2 was completed in 1975 and involved raising the dam approximately 5 feet to elevation 2880 feet.

Phase 3 work was begun in late September 1976 and completed in August of 1977. Phase 3 construction brought the dam to elevation 2890 feet, which was the crest elevation at the time of the July 25, 1980 inspection. The Phase 4 addition, was delayed, and later incorporated in the Phase 5 construction. The Phase 5 addition was under construction at the time of the July 25, 1980 inspection, and was completed in October 1980, which raised the dam to elevation 2925 feet. While the phased construction of the dam has added a total of 75 feet to the 50-foot high starter dam's crest, it has also included approximately 10 feet of elevation difference at the downstream toe, making the total hydraulic height of the Phase 5 dam 135 feet.

A log of the planning and construction of the dam including construction drawings is on file with the chief engineer at the W. R. Grace mill site. Drawings showing the "in-place" construction phases are presented on Plates 4 & 5. A stability analysis for the planned 200 foot high embankment is on file with the owner. During construction of the various phases, Bovay Engineering (for the starter dam) and Harding-Lawson provided engineering services that included observation, consultation and material testing. Records indicate embankment construction met or exceeded design criteria.

Information in the Foundation Investigation and Engineering Analysis, (Reference 6) is cited throughout this Phase I - Dam Safety Report as it contains detailed information regarding the design of the starter dam and discussion of the planned 200 foot high embankment.

CHAPTER 2
INSPECTION AND RECORDS EVALUATION

2.1 HYDRAULICS AND STRUCTURES

2.1.1 Spillway

The spillway (Photos 6 to 13) is located in the east abutment and has been built to the same design criteria as described for the Phase 3 spillway in a June 8, 1976, letter to Robert Foss from Lyle Lewis of Harding - Lawson Associates and in a June 29, 1977, letter to Mr. Purnel Whitehead of W.R. Grace & Co. from Lyle Lewis concerning spillway design data (Reference 7). The spillway design criteria is presented on Plate 7. To our knowledge, the spillway has never been operated.

The spillway consists of an unlined trapezoidal shaped approach channel with a concrete transitional inlet structure which leads to half-sections of 8-foot diameter corrugated metal pipe (CMP) that comprise the spillway chute. Two pipes carrying mine tailings cross the unlined trapezoidial channel only slightly above the invert elevation (Photo 8). These pipes would trap floating debris during periods of spillway use and reduce its capacity. Bank sloughing and deposition of material in this unlined approach channel also represents serious capacity reductions and should be prevented. The entrance elevation of spillway is approximately 5 feet below the dam crest. The trapezoidal channel which extends through the east abutment abruptly transitions to the CMP chute (Photos 6 and 7). The CMP chute meanders along the haul road near the left abutment at an eight percent slope for approximately 500 feet and discharges into an unlined roadside ditch. Discharges enter the natural drainage downstream of the embankment toe. No stilling basin or energy dissipater is provided for at the terminus of the spillway chute (Photos 12 and 13). The CMP is anchored to concrete collars at three points along its length (Photo 10) and is also anchored to the concrete trapezoidal section near the crest. Pipe sections are overlapped and bolted to each other but are not sealed to prevent seepage which could produce piping of foundation materials and cause the chute to fail (Photo 9). This assemblage of the spillway raises doubts that the spillway chute will safely handle any high flows. Because of the spillway location, the ability of the half-round pipe to withstand hydraulic forces under all flow conditions, is an important consideration to the safety of the dam.

There was no floating debris noted in the pond, however, the drainage basin and much of the shoreline is heavily timbered and debris could be easily carried into the pond during high flows. There are no apparent provisions to protect the spillway from floating debris.

A spillway rating (Plate 8) was developed using the HEC-2, Water Surface Profiles (Reference 8). Backwater computations determined that critical depth occurs at the spillway crest entrance (elevation 2,920 feet NGVD). A Mannings "n" of 0.022 was used. The maximum discharge capacity of the spillway with the reservoir at the assumed dam crest elevation 2,925 feet NGVD, was estimated to be 200 cfs or about 17 acre-feet per hour. The two slurry pipes which cross the unlined trapezodial channel were not considered in the capacity analysis.

2.1.2 Outlet/Decant Tower

Flow from the pond under normal operations is controlled by the decant tower. The Phase 5 decant tower is located approximately 1000 feet upstream of the dam (Plate 3) and consists of a 72-inch diameter corrugated metal riser pipe with a 1.7-foot wide rectangular weir entrance with a minimum elevation at 2890.1 feet, NGVD (Photo 3). The maximum weir elevation is the top of the decant tower at approximately 2923.0 feet, NGVD. The purpose of the decant tower is to maintain the level of the settling pond so that solids from the mill tailings are settled out and only the clean surface water is discharged through the decant line. The pond elevation is controlled by stoplogs placed in the decant tower entrance. Stoplogs acting as a weir crest will be installed permanently in stages as tailings accumulate in the pond to maintain several feet of water above the tailings. A log boom (Photo 3) in front of the stoplogs prevents floating debris from clogging the decant tower.

The Phase 5 decant line extends from the tower along the floor of the pond and through the dam embankment discharging approximately 500 feet downstream of the dam. This line consists of 16-inch diameter welded steel pipe. The pipe drains directly from the floor of the decant tower. Mr. Ray stated the portion of the decant line located within the embankment is encased in 1 foot of concrete with two cutoff collars for seepage control. The normal discharge capacity of the decant line was estimated to be 5 cfs and could increase to 18 cfs under unobstructed pressure flow conditions.

2.1.3 Rainy Creek Diversion

Under normal operations Rainy Creek is diverted around the dam. Fleetwood Creek, a tributary to the tailings pond, is not diverted and does enter the reservoir. A 48-inch diameter CMP diversion pipe with an approximate capacity of 100 cfs has been constructed to convey flow from Rainy Creek upstream of the reservoir above the 3,000 foot elevation and divert the flow downstream of the dam (Photo 5). A diversion structure intercepts normal runoff from 60 percent of the drainage basin and discharges it into the diversion pipeline. If inflows were to exceed the diversion pipeline capacity and/or the crest elevation of the flashboard assembly in front of the overflow culvert, then the remaining flow discharges directly into the tailings pond. At the August 13, 1981 inspection, the elevation at the top of the flashboards for the overflow culvert was approximately six inches higher than the water surface in front of the diversion structure, and approximately three feet lower than the structure's crest elevation (Photo 4).

2.1.4. Freeboard

This study indicates the dam overtops during the recommended spillway design flood (SDF) which is the probable maximum flood (see paragraph 2.2.4). Therefore, it has no freeboard. At the time of the July 25, 1980 inspection, the vertical distance from the water level (approximate elevation 2,880 feet) to the dam crest (elevation 2,890 feet) was approximately 10 feet. After the Phase 5 construction, and during the August 13, 1981 inspection, the normal water level was 31 feet below the dam crest. However, the vertical distance between normal water level and the top of the dam will become less

as tailings accumulate in the reservoir. Historically the dam has been raised to provide additional storage when the normal reservoir operating level came to within 10 to 12 feet of the dam crest. The spillway crest on the Phase 5 addition is five feet below the crest of the dam. The dam is located on the southwest end of the pond and the prevailing winds would be directed away from the dam. The effective fetch for wind-generated waves resulting from a north wind is about 2,000 feet and wave run-up on the embankment is estimated to be less than 3 feet. Although the dam will be overtopped by the PMF, the vertical distance between the normal pool elevation and the dam crest is adequate to prevent overtopping of the embankment by wind-generated waves.

2.2 HYDROLOGY, CLIMATOLOGY AND PHYSIOGRAPHY

2.2.1 General

The climate of the area is continental in nature characterized by warm summers and cold winters. Summer temperatures rarely exceed 95°F, and winter temperatures can reach 25 to 30 degrees below 0°F. Winters have few extended extreme cold spells due to periods of warm "chinook" winds.

The Rainy Creek drainage is located between two climatological stations. The Libby 1 N.E. Ranger station site is located 5.5 miles west southwest at an elevation of 2,080 feet NGVD. The Libby station has 74 years of record for temperature and 84 years of record for precipitation. Mean annual precipitation at Libby is 19.4 inches with 37 percent of it occuring in the months November through January and 18 percent falling in the months of May and June. The month having the highest average precipitation is January with 2.42 inches. Temperatures at Libby range from an average of 22.4° Fahrenheit (F) in January to an average of 67°F in July. May and June temperatures average 54°F and 60.3°F, respectively.

A second climatological station is located approximately 5.1 miles east southeast of the basin at Libby Dam. The Libby Dam station is at elevation 2,200 feet NGVD and has 12 years of record for temperature and precipitation.

Average annual precipitation in the Rainy Creek drainage is estimated to be 30 inches per year (Reference 9) and the temperatures would be expected to average 3 to 5 degrees cooler than at Libby.

The Rainy Creek drainage basin above the tailings dam is 9.7 square miles in area and is generally "L" shaped. It is located on a southern exposure of the Purcell Mountains and is primarily forest covered except for the mine area. The basin rises from an elevation of 2,800 feet at the dam to 6,040 feet at Blue Mountain in a stream length of approximately 4.6 miles. There is no gaging station in the basin and the nearest gage downstream is on the Kootenai River at Libby (U.S. Geological Survey Station No. 12303000).

A hydrology report on the Rainy Creek drainage basin was prepared for W.R. Grace and Company in February 1971 by Bovay Engineers, Inc. and indicates the maximum design flow in Rainy Creek at Fleetwood Creek at 200 cfs. The maximum design flow is based on comparisons of maximum discharge of

record, per square mile of drainage area with six nearby and hydrologically similar gaging stations.

2.2.2 Reservoir Storage

In estimating the storage volume of the tailings pond it was assumed that mill tailings deposits had filled the pond to an elevation of 2,873 feet which means the water was approximately 7 feet deep at the time of the inspection. Using the conic method to develop total storage volume for the completed Phase 5 it is estimated that the reservoir would have a surface area of 68.5 acres and a volume of 2,120 acre-feet with the pool at the spillway crest elevation 2,920 feet. Approximately 330 acre-feet of surcharge storage is available in the reservoir between the spillway crest and the dam crest.

Based on estimates of anticipated plant production, tailings fill the reservoir storage at a rate of 135 acre-feet per year. Continued reduction in reservoir floodwater storage by deposition of tailings will seriously affect flood handling potential. Historically the dam has been raised to provide additional storage when the normal reservoir operating level came to within 10 to 12 feet of the dam crest.

2.2.3 Estimated Probable Maximum Flood

The probable maximum flood (PMF) is the flood expected from the most severe combination of critical meterologic and hydrologic conditions that are reasonably possible in the region. An estimate of the PMF was made during this dam safety analysis and was routed through the reservoir.

The probable maximum precipitation (PMP) was developed using a procedure contained in the U.S. Weather Bureau's Hydrometerological Report No. 43 (Reference 10) as updated by U.S. Weather Bureau memorandum dated 9/20/67 (Reference 11). The storm which produces the PMF would be a 6-hour thunderstorm during the period July to August. The July-August thunderstorm PMP produces 6.0 inches of rain in one hour and 8.0 inches of rain in six hours. A minimum loss rate of 0.15 inches per hour was assumed to represent the hydrologic class B soils in the basin and minimum infiltration conditions due to saturated ground. Baseflow was considered to be 110 cfs from Rainy Creek and Fleetwood Creek.

A triangular unit hydrograph for a 10-minute rainfall duration was developed for the 9.7 square mile drainage area using procedures contained in Design of Small Dams (Reference 12). The Soil Conservation Service method of developing a curvilinear fit of the triangular unit hydrograph was used. The hourly increments of the PMP were arranged in a critical time sequence as illustrated in HMR No. 43 page 181 (Reference 10). The 10-minute increments from the greatest two hours were rearranged in the reverse order of the unit hydrograph to produce the greatest possible peak. The PMP was applied to the unit hydrograph by means of the computer program HEC-1 (Reference 13). This estimate of the PMP produced a flood with a peak flow of 43,400 cfs and a volume of 3,770 acre-feet.

2.2.4 Flood Routing

Routing of the probable maximum flood through Zonolite Tailings Pond was performed using the computer program HEC-1 Flood Hydrograph Package (Reference 13). The reservoir routing was started at the minimum elevation of the decant tower (2890.1 feet NGVD) which is the current operating level. This elevation, however, will change as stoplogs are permanently installed to keep water above the tailings. A 100-year 24-hour antecedent storm of 3.4 inches obtained from the National Oceanic and Atmospheric Administration (NOAA) together with Soil Conservation Service (SCS) runoff curve number (CN = 55, forested, class B soil) produced 0.35 inches of runoff. It was assumed that the diversion structure on Rainy Creek had failed allowing the entire runoff from Rainy Creek to join Fleetwood Creek and enter the reservoir. The total volume entering the reservoir would be approximately 180 acre-feet and would raise the reservoir elevation four feet to 2894.0 feet. The decant line discharging at 10 acre-feet per day would leave the reservoir at nearly the same elevation (2894.0 feet NGVD). This elevation was used to start the PMF routing.

Information from the owner regarding reservoir levels indicates that the drainage basin which is not controlled by the Rainy Creek diversion has never seriously contributed to raising the pond elevation. Only the installation of flashboards in the decant tower causes changes in pond elevation. There is no record of either Phase 3 or Phase 5 spillway use.

Routings indicate that the dam overtops during the PMF when approximately 55 percent of the total flood volume enters the reservoir. Routings were made of lesser hypothetical floods than the PMF to determine the magnitude of floods the dam can contain. The hypothetical hydrographs are obtained by applying percentages to the PMF hydrograph ordinates. A flood with a hydrograph having ordinates corresponding to 45 percent of the PMF ordinates is just controlled by the project. Larger floods would overtop the dam. Depletion of water storage volume by adding mill tailings would further reduce project flood handling capabilities.

2.3 GEOTECHNICAL EVALUATION

The following geotechnical evaluation is based on field inspections of the project, examination of the referenced reports, plans, and specifications, and discussion of the construction history with Bob Foss and Mike Ray, Chief Engineers for Zonolite Operations of W.R. Grace and Company, and Lyle Lewis, consulting design engineer, Harding-Lawson Associates.

2.3.1 Dam

The Zonolite Tailings Dam, is a homogenous fill resulting from several phases of incremental construction. The different phases of incremental construction of the embankment are shown on Plate 5. Phases 4 and 5 were constructed simultaneously. The elevation at the base of the dam is about 2,790 feet. The initial starter dam crest elevation was 2,850 feet and successive increments were added at elevations 2,875, 2,880, and 2,890. The phase of construction ongoing at the time of the July 25, 1980 inspection increased the dam height 35 feet to elevation 2,925 feet.

Each of the increments were added to the dam embankment using a downstream method of construction with slopes of 1V on 2H for upstream and downstream faces, and intermediate benches on the downstream face as shown on Plate 5. Table 1 shows a summary of dam height, crest length, and crest width for each successive increment of dam height. The final downstream slope configuration for the structure at elevation 2,925 feet NGVD including erosion benches is shown on Plate 5 and Photos 6, 14, 15 and 16.

Table 1: Summary of general dimensions for incremental structures.[1/]

ELEVATION (Feet)	DAM HEIGHT (Feet)	CREST WIDTH (Feet)	CREST LENGTH (Feet)
2850 Starter Dam	50	40	840
2875 Phase 1	75	40	975
2880 Phase 2	80	35	995
2890 Phase 3	95	22	1002
2899 Phase 4*	105	80	1055
2925 Phase 5	135	40	1154

[1/] Crest lengths and widths scaled from W.R. Grace & Co. Construction Products Division Drawing No. 40-1009 as revised for each construction phase (Plate 4).

*incorporated in Phase 5 construction.

The materials used to construct the embankment are old mill tailings obtained from a stock pile on the east side of the tailings retention pond area, soils stripped from the abutment areas, and gravel from the location of the old mill pond downstream from the existing structure. Memos and letters in the Harding-Lawson Associates job file indicate that embankment fill was placed in three- to six-inch lifts and compacted to 95 percent relative compaction as determined by laboratory test procedures and periodic field density tests. Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same material, as determined by the ASTM D1557-70(C) laboratory test procedures. Records of extensive field density tests during construction phases (thru Phase 3) show relative compaction of 95 percent or more (Reference 7). Discussion with the design consultant indicates that field density tests during Phase 5 construction also conformed with design criteria.

The main bulk of the embankment consists of mill tailings. Preconstruction tests show the mill tailings to consist of greenish gray gravelly sand with a dry density of about 138 pounds per cubic foot (S.G.=3.1), a permeability of 1×10^{-3} feet per day at 95 percent relative compaction, an internal angle of friction (ϕ) of 20°, and cohesion (c) of 4000 psf, (Reference 6 and Reference 14). Strength parameters of ϕ and c are based on consolidated-undrained triaxial tests with pore pressure measurements. Maximum dry density data for mill tailings provided for the field density tests range from 144 to 149 pounds per cubic foot. Tests show surficial material on the abutments to be gray silty sandy gravel and sandy gravel

that mix to a dry density of 128 pounds per cubic foot and exhibit a permeability of 2×10^{-5} feet per day at 95 percent relative compaction (Reference 6).

Inspection of the embankment did not reveal any sign of cracking, differential settlement, misalignment, or slope failure. A review of the extensive construction inspection records (Reference 7) and discussions with the W.R. Grace Engineer indicate that there have been no stability or consolidation problems with the embankment.

There is no protective shell or riprap on either face of the embankment. However, the coarse-grained mill tailings slurry discharged at the upstream face of the embankment, prohibits serious wave erosion of the embankment face (Photo 17). The downstream face of the dam has been under intermittent construction since 1971. There was evidence on both inspections that rain storms had caused accelerated erosion on the downstream dam slope (Photo 15). Bob Foss stated that rill erosion between benches on the downstream face demands seasonal maintenance by face dressing. A new surface drainage system consisting of half-sections of 42-inch diameter CMP has been constructed in the left abutment (Photo 14). This system drains the bench cuts on the downstream face of the Phase 5 addition. Erosion on the dam face does not pose a hazard to the dam so long as timely maintenance is performed and erosion control measures such as bench cuts or erosion terraces are used. During the PMF, rainfall intensity of 8 inches in 6 hours may cause extensive erosion on the dam's face.

2.3.2 Foundation Conditions, Seepage, and Drainage

The foundation of the Zonolite Tailings Dam, as determined by test borings, test pits, resistivity soundings, and seismic velocity measurements (Reference 6) consists of 20 to 40 feet of sand and gravel outwash with interbedded lenses of fine sand and silt all resting on weathered pyroxenite bedrock. Unsuitable materials were stripped from the foundation. Embankment fill was placed on dense gravelly soils containing abundant cobbles and boulders following foundation stripping. Foundation stripping and preparation was accomplished in phases corresponding to the phases of embankment construction.

Abutment preparation prior to placement of compacted fill consisted of constructing bench cuts 4 to 6 feet wide into the abutments. Bench cuts on the left abutment are in weathered, friable magnetite pyroxenite as observed during the July 25, 1980 inspection and documented in construction inspection memos. Bench cuts on the right abutment required 10 to 15 feet of excavation to remove loose slope debris so that the bench cuts key into the dense glacial soils. Construction inspection memos and as-built drawings for the construction increment bringing the dam crest to elevation 2,890 feet NGVD, describe right abutment bench cuts keyed into glacial soils below elevation 2,860 feet and bedrock above that elevation. At the time of the July 25, 1980 safety inspection, the materials exposed by bench cuts on the right abutment above elevation 2,890 feet consisted of firm glacial soils.

Construction inspection memos and letters (Reference 7) beginning October 24, 1972 describe the presence of an adit in the right abutment in the Rainy Creek ditch cut below elevation 2,875 feet. Discussions with Lyle Lewis of Harding-Lawson Associates revealed the adit was backfilled and drain pipes were installed to control seepage. The absence of seepage in the right abutment contact area and the width of the embankment fill in the reported location of the adit suggest the adit does not pose a threat to the abutment integrity.

Drainage installations employed at the Zonolite Tailings Dam consist of a chimney drain in the original starter dam and foundation drains consisting of perforated concrete pipe bedded in pervious aggregate. The location of foundation drains and the location of the chimney drain, which was not extended beyond the starter dam, are shown on Plates 4 and 5. The foundation drainage system presently consists of two cross drains and essentially seven lateral drains (Photo 12 to 25).

The starter dam was constructed immediately downstream of an older existing dam, which was reported to have a crest elevation of 2830 feet. A foundation cross drain was placed roughly from abutment to abutment and located between the upstream toe of the starter dam and the older existing dam. The cross drain is indicated in the foundation drain pattern on Plate 4 and is composed of a 10 foot wide by one foot thick bed of select drain material. This upstream toe drain is covered by compacted mill tailings used to fill the space between the starter dam and the old existing dam. The so called toe drain discharges into two of the lateral drains under the starter dam embankment.

Construction memos (Reference 7) indicate that toe drains 18 to 24 inches wide and 36 to 48 inches deep were placed at the toe of the starter dam and the first addition to the dam as trench drains to intercept seepage from the natural foundation gravels observed during construction. The toe drains were limited to the abutment portion of the starter dam downstream toe. Foundation stripping for the first addition to the starter dam revealed a spring in the foundation. The spring was excavated and filled with pervious gravel and is drained by the 14 inch steel pipe shown on Plate 4 and Photos 19, 21 and 22.

The foundation drainage system shown on Plate 4 was extended and in some areas expanded as part of the Phase 5 addition. Five perforated concrete drain pipes bedded in gravel were inspected (July 80 and Aug. 81) and each revealed clear water either seeping from the drain pipe or bedding material (Photos 19 to 25). A wetted front along the downstream toe was observed during the August 13, 1981 inspection and extended between all drains at approximate elevation equal to the flow line of the drains. The two 10-inch drain pipes closest to the spillway shown on Plate 4 do not extend to the embankment toe. These two drain pipes are buried but are scheduled to be uncovered by the owner. The 14 inch steel pipe draining the spring in the foundation was discharging flow estimated to be 100 to 200 gpm (Photo 22).

Piezometer monitoring data are available for each phase of dam construction. Five piezometers in the completed Phase 5 addition are shown on

Photo 17 and are located on Plate 5. These piezometers are merely vertical extensions of those that were in place for Phase 3. Only seven readings of the Phase 3 piezometers were recorded in a 29 month period between December 1977 and May 1980. Readings of the water level in the open tube piezometers should be correlated to tailings pond levels but records do not indicate such data is being kept. Although limited information has been obtained since the completion of Phase 5, there continues to be no evidence of significant water levels above the foundation.

Additional piezometer data was collected during the August 13, 1981 follow-up inspection. Measurements included elevations at each piezometer casing top, depths to water from the piezometer casing tops, and total piezometer depths as measured from the casing tops. Table 2 shows elevation and depth measurements conducted on August 13, 1981 and compares measured bottom elevations for each of the piezometers to bottom elevations shown on the as-built drawings (Plate 5).

TABLE 2: Measured piezometer elevations and depths compared to as-built data.

Piezometer No. (Plate 5)	Measured Top Casing Elev.[1/] (Feet)	Measured Piezometer Depth (Feet)	Measured Bottom Elev. (Feet)	As-Built Bottom Elev. (Feet)	Difference From As-Built (Feet)
#1	2,920	104	2,816	2,807	9
#2	2,920	121	2,799	2,790	9
#3	2,920	60	2,860	2,795	65
#4	2,921	106	2,815	2,806	9
#5	2,921	104	2,817	2,811	6

1/ Measured elevations based on assumed crest elevation 2,925 at centerline of dam crest at midpoint between abutments and August 13, 1981 survey. Elevations and depths rounded to nearest foot.

Table 2 shows that measured elevations for the bottoms of the piezometers as of August 13, 1981 are higher than those shown on the as-built (Plate 5). Piezometer #3 in particular is 65 feet shallower than originally constructed. W. R. Grace and Company Chief Engineer for Zonolite Operation, Michael Ray, states that there are indications that vandals have been dropping rocks and loose earth down the piezometers. Measurements in piezometer #3 on August 13, 1981 showed considerable dirt sticking to condensation on the side of the casing beginning at about 34 feet from the top of casing and persisting to the bottom at above 60 feet below the casing top. The foregoing observations suggest that the differences between measured and as-built piezometer depths and bottom hole elevations (Table 2) may possibly be attributed to rocks and earth dropped into the piezometers.

Table 3 shows the depths to water and water surface elevations measured in the piezometers on August 13, 1981. Water surface elevations are compared to the as-built foundation elevation shown on Plate 5. Data on Table 3 show a phreatic surface in the lowermost 22 feet and 15 feet of the dam embankment at piezometers #1 and #2, respectively. Both piezometers #1 and #2 bottom out in the embankment fill without penetrating foundation material. Piezometer #3 is too plugged to yield useful data. Piezometers #4 and #5 both penetrate foundation materials and indicate that the phreatic surface in the foundation at these locations is at a lower elevation than the base of the embankment. The data shown on Table 3 indicate that the foundation drains and the pervious alluvial foundation materials are effectively controlling embankment and foundation seepage within conservatively safe levels.

TABLE 3: Water surface elevations measured in piezometers on August 13, 1981.

Piezometer No.	Measured Top Casing Elev.[1/] (Feet)	Measured Depth to Water[2/] (Feet)	Water Surface Elev.[3/] (Feet)	As-Built Foundation Elev. (Feet)	Pond Elev.[3/] August 13, 1981 (Feet)
#1	2,920	102.03	2,818	2,796	2,894
#2	2,920	114.76	2,805	2,790	2,894
#3	2,920	DRY	DRY	2,796	2,894
#4	2,921	DRY	DRY	2,812	2,894
#5	2,921	103.89	2,816	2,841	2,894

1/ Measured elevations based on assumed crest elev. 2,925 at centerline of dam crest at midpoint between abutments and August 13, 1981 survey. Elevations and depths rounded to nearest foot.
2/ Water levels measured by steel tape and chalk method to ±0.01 feet.
3/ Water surface elevation rounded to nearest foot.

2.3.3 Stability

A design stability analysis for the proposed 200-foot high tailings embankment is on file with the owner. Since all lower dams are similar to the planned 200-foot high embankment, consist of homogenous coarse tailings, and have 1V on 2H upstream and downstream slopes and a 20-foot minimum crest width, no stability analyses were performed for individual construction phases. The design stability analysis used an assumed phreatic surface and shear strength data developed from testing of embankment mill-tailing materials.

Results of the field testing through Phase 3 construction were reviewed and are in conformance with the design criteria as established by Harding-Lawson Associates and are in conformance with preconstruction values used in the stability analysis. The recent Phase 5 construction was inspected by a Harding-Lawson engineer and Mr. Lewis reports that field density tests

performed during construction show densities of at least 95% of optimum were achieved. Considering the low phreatic surface (as indicated by piezometers) compared to the phreatic surface used in the stability analysis, it is our judgment that Zonolite Tailings Dam conforms with the Recommended Guidelines stability criteria.

2.4 PROJECT OPERATION AND MAINTENANCE

Information on operations and maintenance was obtained from discussions with Mr. Robert Foss and Mr. Michael Ray, Chief Engineers for W.R. Grace and Company, Zonolite Operations. There is no formal operations and maintenance plan for the project.

2.4.1 Dam

The dam is located at the Zonolite mining operation site and is essentially under constant observation. Mr. Ray is responsible for the on-going maintenance and construction program for the dam and maintains a construction log (Reference 7) of all construction activity and data. Because the dam is being constructed in a phased program under the direction of an engineering design consultant, the embankment is essentially receiving major maintenance attention on a regular basis. Since 1971 when the starter dam was constructed, major construction modifications to the embankment have been performed every 2 to 3 years. Annual maintenance is also performed on the dam which includes dressing the slopes each spring or on an as-needed basis to repair minor erosional problems from surface runoff. Trees and brush are not allowed to become established on the embankment and burrowing animals are not of concern.

2.4.2 Reservoir

Zonolite Tailings Pond is a settling basin for the treatment of slurry mill tailings from the mining operation. The level of the pond is controlled by the decant tower and is maintained at an elevation such that the water from the surface meets necessary discharge quality requirements. As the sediments in the lake build up, storage volume and detention times are decreased and the pond's operating level is raised by placing stoplogs in the decant tower to meet water quality discharge requirements. Historically, the dam is raised when the normal reservoir operating level rises to within 10 to 12 feet of the dam's crest.

2.4.3 Warning Plan

There is no formal warning plan for use in the event of impending dam failure. However, because the project is occupied 24 hours a day early warning of unsafe conditions is probable.

CHAPTER 3
FINDINGS AND RECOMMENDATIONS

3.1 FINDINGS

Visual inspections of the dam, review of construction documents, and analysis of the project in terms of the recommended guidelines' performance standards, resulted in the following findings.

3.1.1 Size, Hazard Classification, and Safety Evaluation

The 135-foot-high Zonolite Tailings Dam could impound a maximum 2450 acre-feet of water with the pond at the crest of the dam and tailings at assumed elevation 2873.0 feet. Water storage decreases as tailings storage increases. In accordance with inspection guidelines, Zonoline Tailings Dam is large in size with a high downstream hazard potential rating. The recommended spillway design flood (SDF) for this project is 100 percent of the PMF. This dam safety study indicates that the project, with maximum water storage of 2450 acre-feet, controls a flood with hydrograph ordinates equal to approximately 45 percent of the PMF hydrograph ordinates. Larger floods will overtop the dam. The steady decrease of flood storage by tailings deposition will lower the flood routing capability. The dam is constructed of materials that would quickly erode and rapidly fail if overtopped by flood waters. Such failure could endanger life and property downstream. Because the project cannot safely handle the recommended SDF which is the full PMF, Zonolite Tailings Dam does not conform with inspection guideline hydrologic and hydraulic criteria.

3.1.2 Embankment

Zonolite Tailings Pond is impounded by an earthfill embankment dam. Since the construction of the starter dam in 1971 the embankment has been raised several times with the most recent being Phase 5. The additions to the embankment have been placed using downstream method of construction, i.e. the additional embankment is placed entirely on the downstream and crest portion of the dam which results in a downstream adjustment of the crest alignment.

The crest on the Phase 5 addition exhibits nearly a 2.0 feet range in elevation along its length. The range in elevation measured along a baseline, offset approximately 13 feet downstream of the centerline. The reason for the range in elevation is unknown as the dam is not equipped with settlement monitors and no "as built" design drawings are available. Annual inspections should monitor this condition. No visual signs of cracking, settlement, or slope instability were observed. Rill erosion on the embankment slopes was evident during both inspections. Seasonal maintenance of the slope erosion is required. Owner records and discussions with project personnel indicates embankment stability has not been a problem. There is no slope protection on the upstream slope of the dam, however, there is also no indication of wave erosion. Prevailing winds would be directed away from the embankment. The vertical distance from the normal reservoir level to the dam crest is adequate to prevent wind-generated waves from overtopping the embankment.

Drains placed in the embankment foundation have been extended to accommodate the additional fill from the phased construction activity. Only clear water has been observed seeping from the drain pipe and bedding material of the five perforated concrete drains. The drain nearest the northwest abutment which was dry in July 1980 was flowing in August 1981. In addition to the foundation drains, a 14-inch diameter steel pipe draining a spring in the foundation was discharging an estimated flow of 100 to 200 gpm. There was no evidence of a wetted front or seepage at the downstream toe of the embankment on July 25, 1980, however, during the August 13, 1981 inspection a wetted front equal in elevation to the flow line in the drains was observed.

A stability analysis for the proposed 200-foot high embankment was prepared in 1971 and is on file with the owner. The analysis was based on preconstruction data that have since been verified by field and laboratory test conducted during all phases of completed construction. Material strength results for all constructed phases of the dam are on file in the office of Harding-Lawson Associates. Piezometers have been installed and monitored with each phased addition to the embankment, however, attention to the changing tailings pond level should be noted with each reading.

Review of design stability analyses indicates that the stability analysis is adequate. In our judgement Zonolite Tailings Dam conforms with the Recommended Guidelines stability criteria.

3.1.3 Spillway and Reservoir Capacity

The reservoir has a surface area of about 68.5 acres and a water storage capacity of 2,120 acre-feet at the spillway crest, elevation 2,920 feet NGVD, assuming tailings to elevation 2873.0 feet. Approximately 330 acre-feet of surcharge storage is available between the elevation of the spillway and the crest of the dam. The discharge capacity of the spillway with the reservoir at the dam crest, is about 200 c.f.s. Because the structural integrity of the spillway is questionable, it is uncertain that the structure can safely contain the design flows without adversely affecting embankment safety.

3.1.4 Outlet Works/Decant Tower

The decant tower provides the normal operational releases from the tailings pond. Inspection of the Phase 5 tower and control facilities shows them to be in good operating condition. A welded steel decant line which rests on the pond floor and extends through the embankment is small and thus inaccessable for inspection without special equipment.

3.1.5 Operations and Maintenance

Zonolite Tailings Pond is operated as a settling basin for the milling operation at the mine site. Flow from Rainy Creek is diverted around the dam so under normal operating conditions the inflow to the pond is from the tailings discharge and from Fleetwood Creek. The pond elevation is regulated in small incremental rises to control the quality of discharges.

The embankment is essentially in a state of constant inspection and maintenance. Equipment necessary for dam maintenance is available at the mine site and maintenance is performed on an as-needed basis. There is no formal downstream warning plan for use in the event of impending dam failure.

3.2 RECOMMENDATIONS

Due to storage between normal pool and dam crest, the present project provides a degree of flood protection to the downstream area. The intent of report recommendations is to maintain or improve project safety, if feasible, without decreasing this flood protection.

The findings suggest that high priority be given the following recommendations:

1. Immediately develop, implement, and periodically test an emergency warning plan for use in the event of impending embankment overtopping or structural failure.

2. Periodically test the decant line in the section which passes through the embankment for possible leaks which could threaten the embankment. Repair if required.

3. Conduct more detailed hydrologic and hydraulic routing studies to better determine the downstream hazard potential and to establish the safe minimum flood storage volume and spillway requirement. Studies should take into account the continually decreasing water storage volume as tailings are deposited and the effect of sudden release on the downstream environment. Monitor tailings accumulation and periodically evaluate the available flood storage volume. Evaluate the structural adequacy of the spillway under a full range of possible flow conditions. Remove the two tailings transfer pipes from the spillway approach section. Modify the project as studies indicate.

4. Continue to conduct inspections of the dam on an annual basis by engineers experienced in dam design and construction, continue to monitor and evaluate piezometers, foundation and toe drains and maintain a construction log of all additions and modifications to the project. Add piezometers during construction as required to define the phreatic surface in the dam. Any changes in the position of the phreatic surface should be fully evaluated with respect to its affect on stability. All existing and future piezometers should be sealed and capped to prevent tampering by vandals. Existing piezometers should be cleaned and unplugged or replaced if reliable and useful data cannot be collected.

Prior to performing engineering studies or remedial construction, coordinate with applicable State and Federal agencies to insure compliance with all pertinent laws and regulations.

REFERENCES

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2. Fenneman, N.M., 1931, Physiography of Western United States: New York, McGraw-Hill Book Co., Inc., 534 p.

3. Johns, W.M., 1970 Geology and Mineral Deposits of Lincoln and Flathead Counties, Montana: Montana Bureau of Mines and Geology Bulletin 79, 182 p.

4. Boettcher, A.L., 1966, The Rainy Creek igneous complex near Libby, Montana: Unpub, Ph.D Dissert., Pennsylvania State Univ., 155 p.

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6. Lewis, L.E. and Lawson, R.T., 1971a, Foundation Investigation and Engineering Analysis, Tailings Dam, W.R. Grace & Company, Construction Products Division, near Libby, Montana: Harding, Miller, Lawson & Associates, 55 Mitchell Boulevard, P.O. Box 3030, San Rafael, California; prepared for Bovay Engineers, Inc., Spokane, Washington.

7. Harding - Lawson Associates, 1971 to 1977, Job file office memoranda and letters pertaining to construction inspection and supervision including field density test records; memoranda and letters for period September 15, 1971, to October 13, 1977; logs of Tailings Dam prepared for Construction Products Division, W.R. Grace & Company, Libby, Montana 59923.

8. U.S. Department of Army, Corps of Engineers Hydrologic Engineering Center; "HEC-2 Water Surface Profiles", Generalized Computer Program Users Manual with Supplement; Nov. 76.

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10. U.S. Weather Bureau, Hydrometeorological Report No. 43, "Probable Maximum Precipitation, Northwest States," Washington, D.C., 1966.

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13. U.S. Army Corps of Engineers, Hydraulic Engineering Center, "Flood Hydrograph Package (HEC-1) Users Manual for Dam Safety Investigations", Davis, Calif., Department of the Army 1978, 93.

14. Lewis, L.E., 1971, Letter to Gene McKay, Bovay Engineers, Inc., re: Preliminary laboratory data, Tailings Dam, near Libby, Montana: Harding, Miller Lawson & Associates, January 21, 1971.

15. Lewis, L.E., 1971, Letter to Gene McKay, Bovay Engineers, Inc. Subject: Preliminary laboratory data, Tailings Dam, near Libby, Montana; with Particle Size Analysis, Compaction Test Data, Static Triaxial Shear Test Data, and Dynamic Shear Test Data: Harding, Miller, Lawson & Associates, January 21, 1971, 55 Mitchell Boulevard, San Rafael, California 94902.

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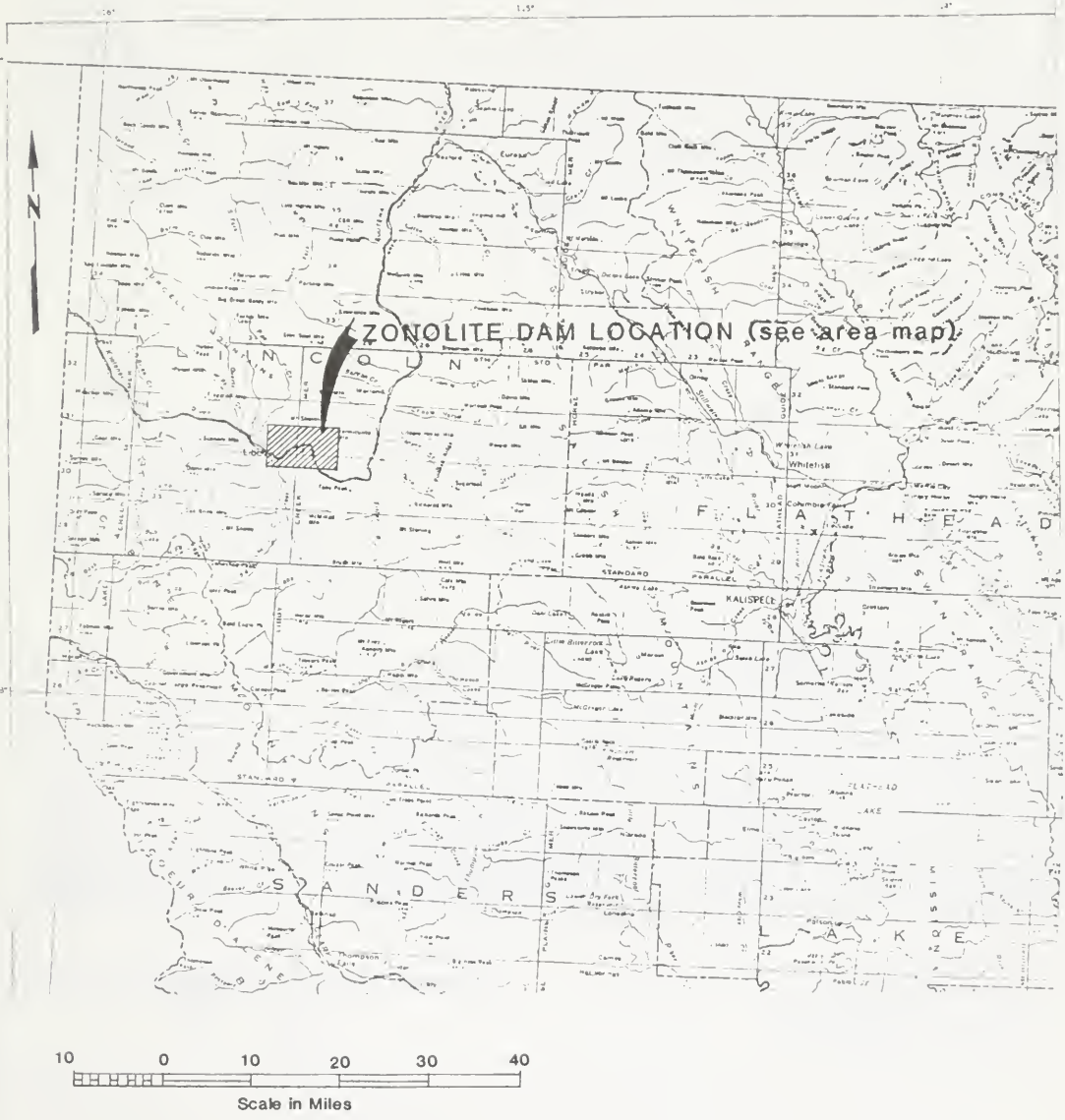
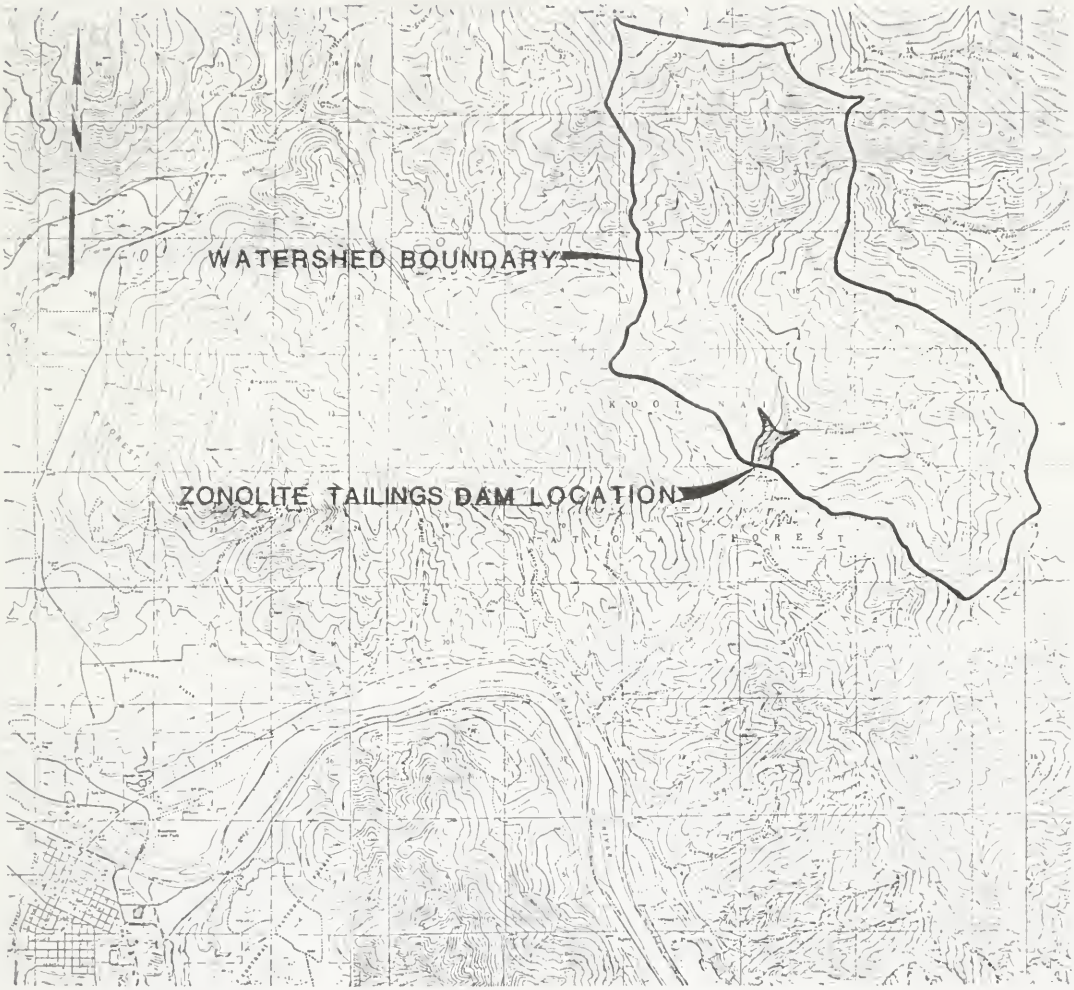


VICINTY MAP

PLATE 1

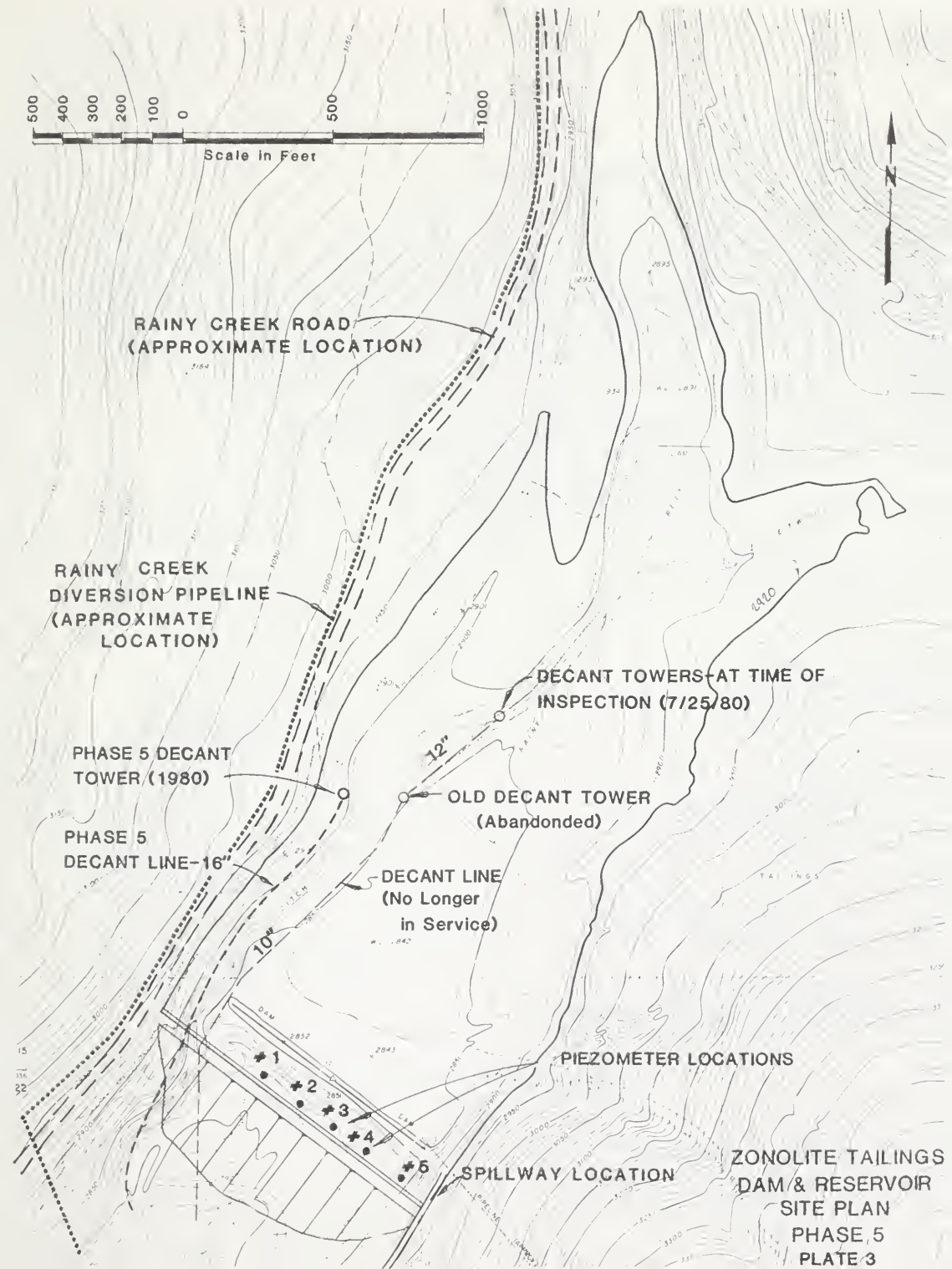


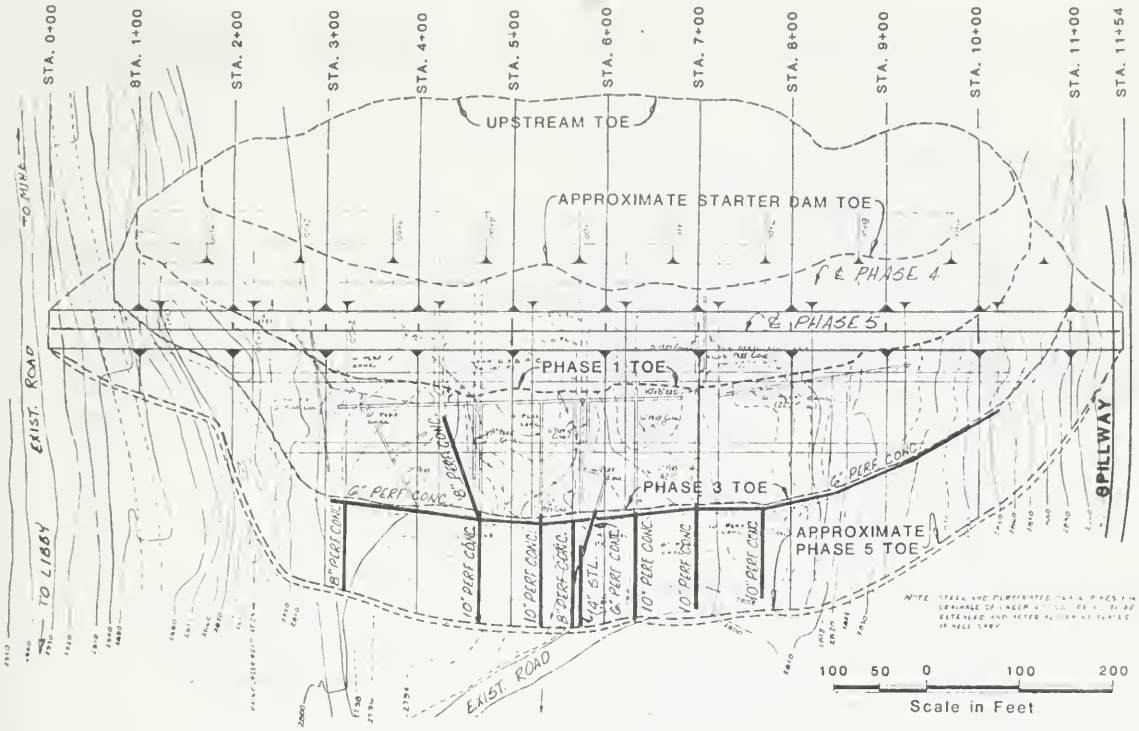
1 0 1 2 3

Scale in Miles

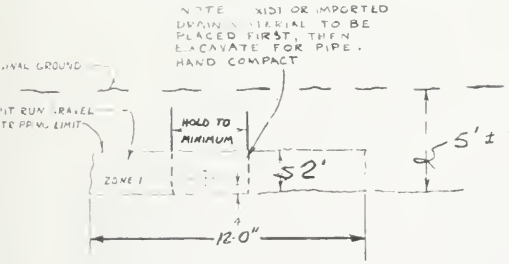
Source of Base: USGS Quadrangles–
VERMICULITE MOUNTAIN (1963),
LIBBY (1963),
ALEXANDER MOUNTAIN (1963)

ZONOLITE TAILINGS DAM
AREA MAP
PLATE 2


500 400 300 200 100 0 500 1000
Scale in Feet
N
RAINY CREEK ROAD (APPROXIMATE LOCATION)
RAINY CREEK DIVERSION PIPELINE (APPROXIMATE LOCATION)
DECANT TOWERS-AT TIME OF INSPECTION (7/25/80)
PHASE 5 DECANT TOWER (1980)
OLD DECANT TOWER (Abandonded)
PHASE 5 DECANT LINE-16"
DECANT LINE (No Longer in Service)
12"
10"
PIEZOMETER LOCATIONS
#1
#2
#3
#4
#5
SPILLWAY LOCATION
ZONOLITE TAILINGS DAM & RESERVOIR SITE PLAN PHASE 5
PLATE 3

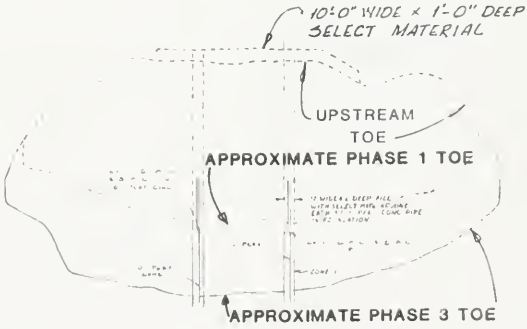


PLAN



TYPICAL DRAIN SECTION

NOT TO SCALE



TYPICAL
FOUNDATION DRAIN PATTERN

NOT TO SCALE

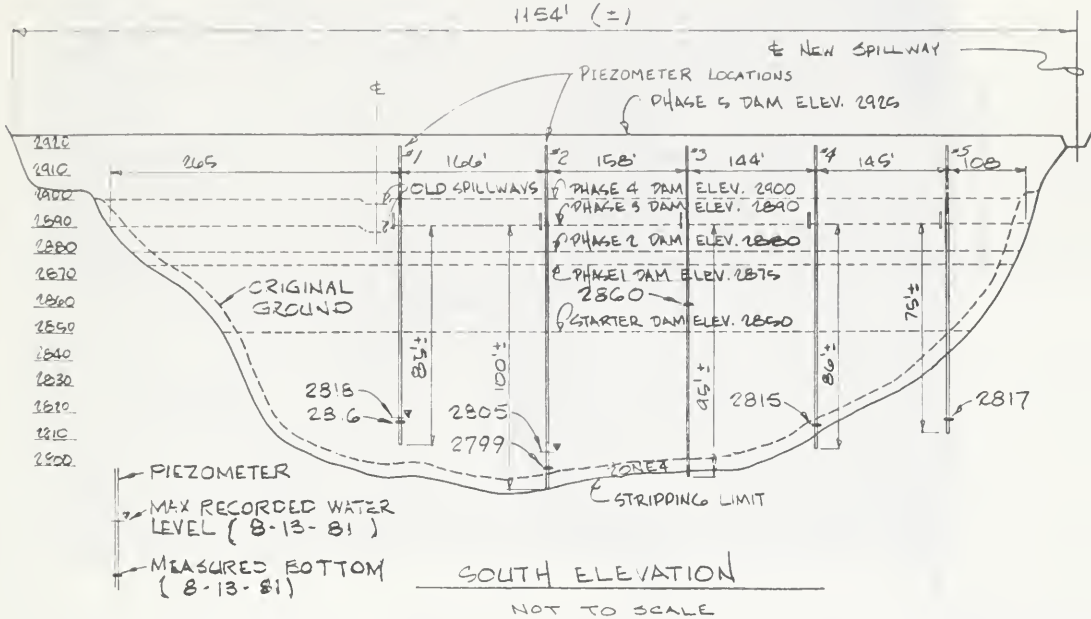
Source: W.R. GRACE & CO. Drawing No. 40-4009 4 (TAILINGS IMPOUNDMENT-PHASE 4)

Drawing No. 40-4009 4 (TAILINGS IMPOUNDMENT-PHASE 5)

ZONOLITE TAILINGS DAM

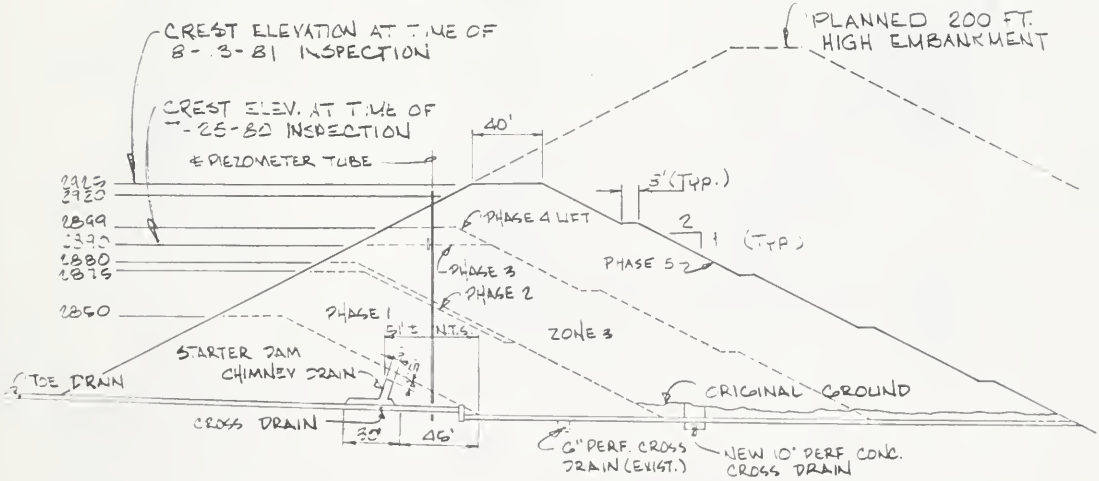
PLAN VIEW

PLATE 4



SOUTH ELEVATION

NOT TO SCALE

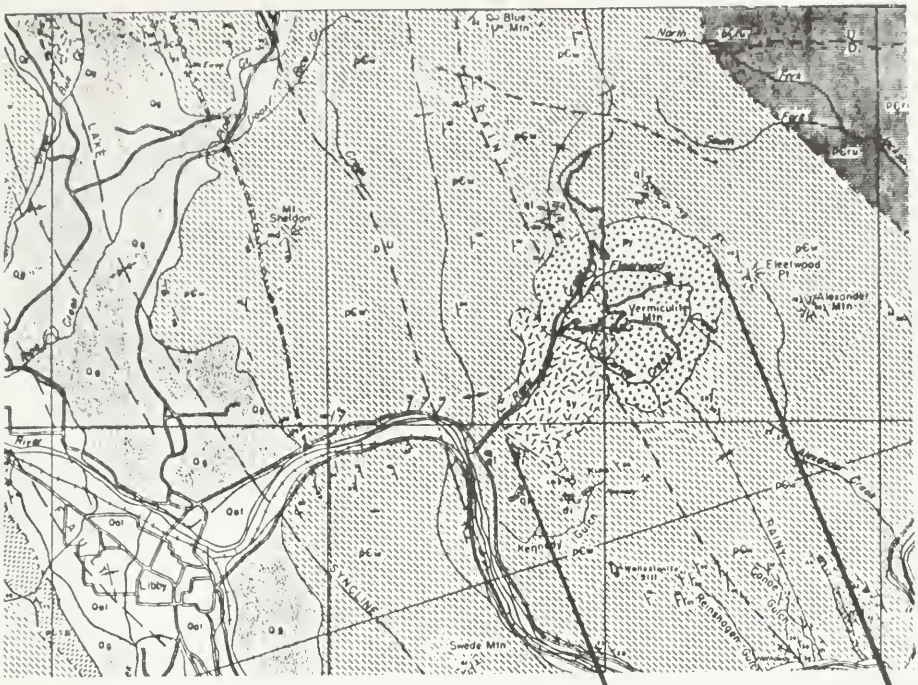


TYPICAL CROSS SECTION

NOT TO SCALE

ZONOLITE TAILINGS DAM
PHASE 5 ADDITION

PLATE 5



SEDIMENTARY ROCKS

Striped Peak Formation

Gray and grayish-red sericitic quartzite and gray-red purple argillite. Some interbedded gray-green laminated argillite near center. Ripple-marked and mud-cracked strata common; some casts of salt crystals.

Kavalli undifferentiated

Medium- to coarse-grained medium- to light-gray quartzite, quartzitic argillite, and argillite; some interbedded and laminated green-gray quartzitic argillite weathering gray. In the Flathead Lake region grayish-red outpbe, gray-red, and grayish-green argillite and local lenticular calcareous quartzitic greenish-gray argillite. Upper Kavalli mapped as pCr$_3$.

Wallace Formation

Gray, green-gray, and yellow calcareous and dolomitic sandy argillite; discontinuous dolomite and magnesian limestone beds are common near the middle of the formation. Grayish-red shale and micaceous quartzite locally present near the top. Mud cracks and ripple marks common.

Quaternary glacial deposits

Undifferentiated lacustrine silt, clay, sand, gravel, and alluvial fan material

Alluvium

Recent gravel, sand and silt. Some Pleistocene glacial outwash

IGNEOUS ROCKS

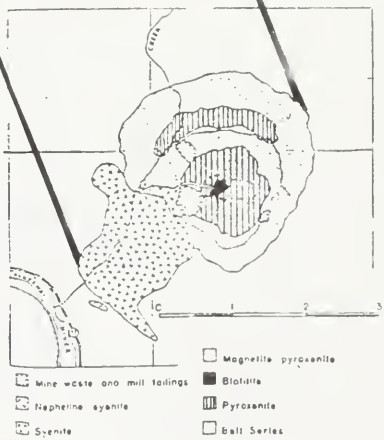
Diorite dikes

Syenite stocks

Quartz latite porphyry dikes

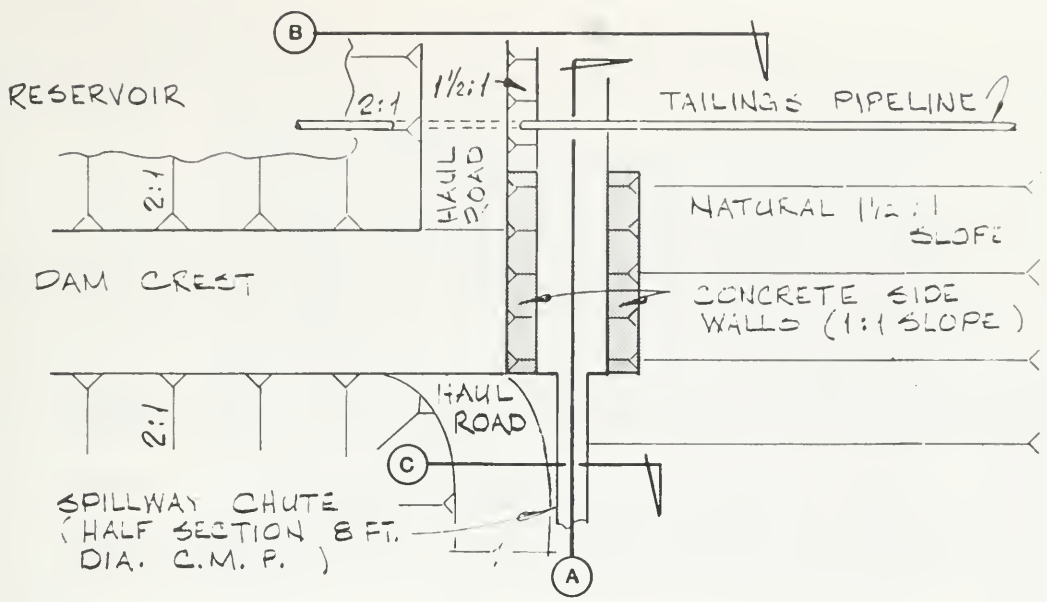
Basic stock composed of pyroxenite

Metadiorite–dark-green gray and gray-black dikes, sills, and stocks mapped by others as gabbro, hornblende gabbro, amphibolite, and metadiorite

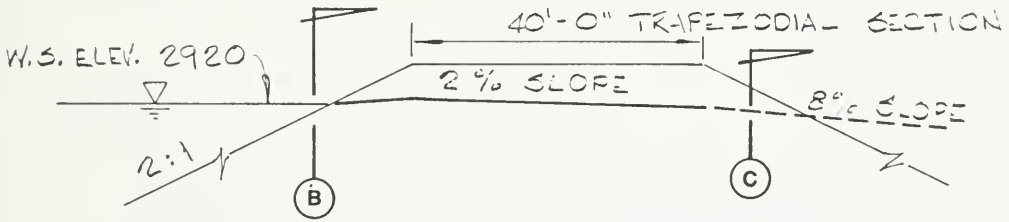


GEOLOGY MAP

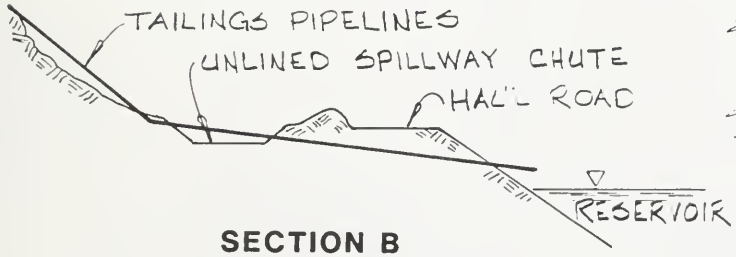
PLATE 6



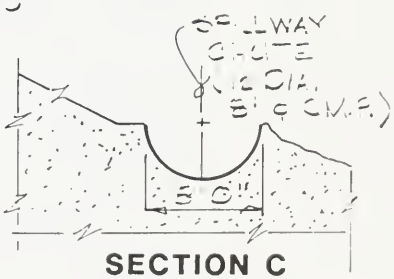
PLAN VIEW-SPILLWAY EAST ABUTMENT
SCALE: 1" = 40'



SECTION A-SPILLWAY PROFILE
SCALE: 1" = 40'

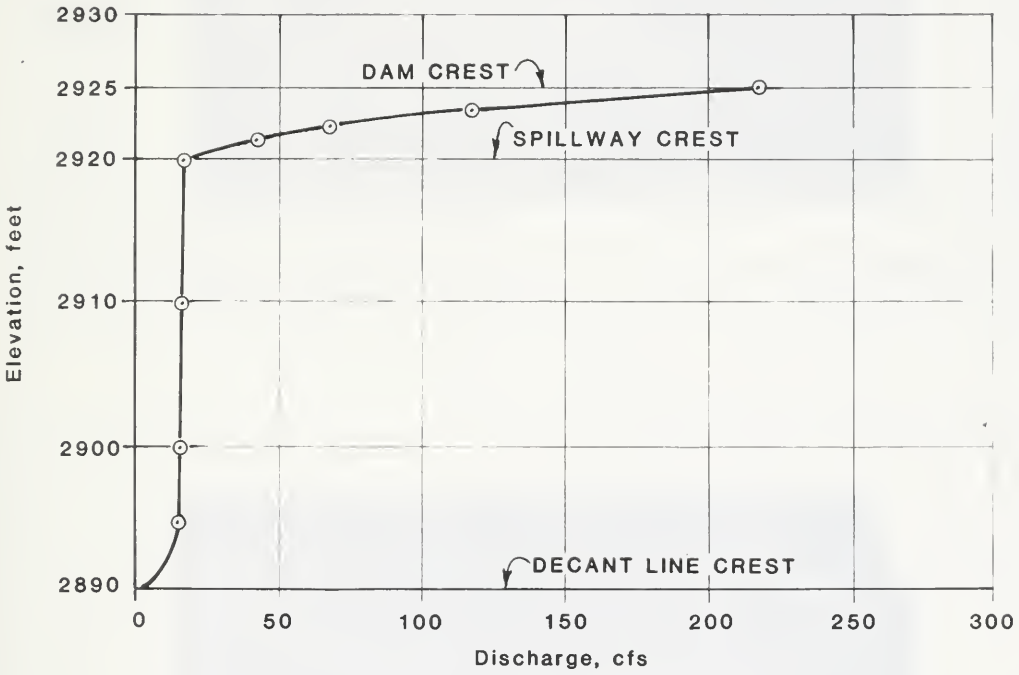


SECTION B
SCALE: 1" = 40'



SECTION C
NO SCALE

ZONOLITE TAILINGS DAM
SPILLWAY DESIGN
PHASE 5 TYPICAL SECTIONS
PLATE 7



ZONOLITE TAILINGS DAM
COMBINED DECANT LINE
&
SPILLWAY RATING CURVE
PLATE 8

Photo 1 Aerial View of Zonolite Tailings Dam Looking Upstream

Photo 2 Aerial View of Zonolite Dam and Area Immediately Downstream (7-25-80)

Photo 3 Phase 5 - Decant Tower
(7-81)

Photo 4

Rainy Creek Diversion Structure
Foreground: Entrance to Rainy Creek -
Background: Entrance to Diversion Pipeline
(8-13-81)

Photo 5 Rainy Creek Diversion Pipeline Along Rainy
Creek Road (7-81)

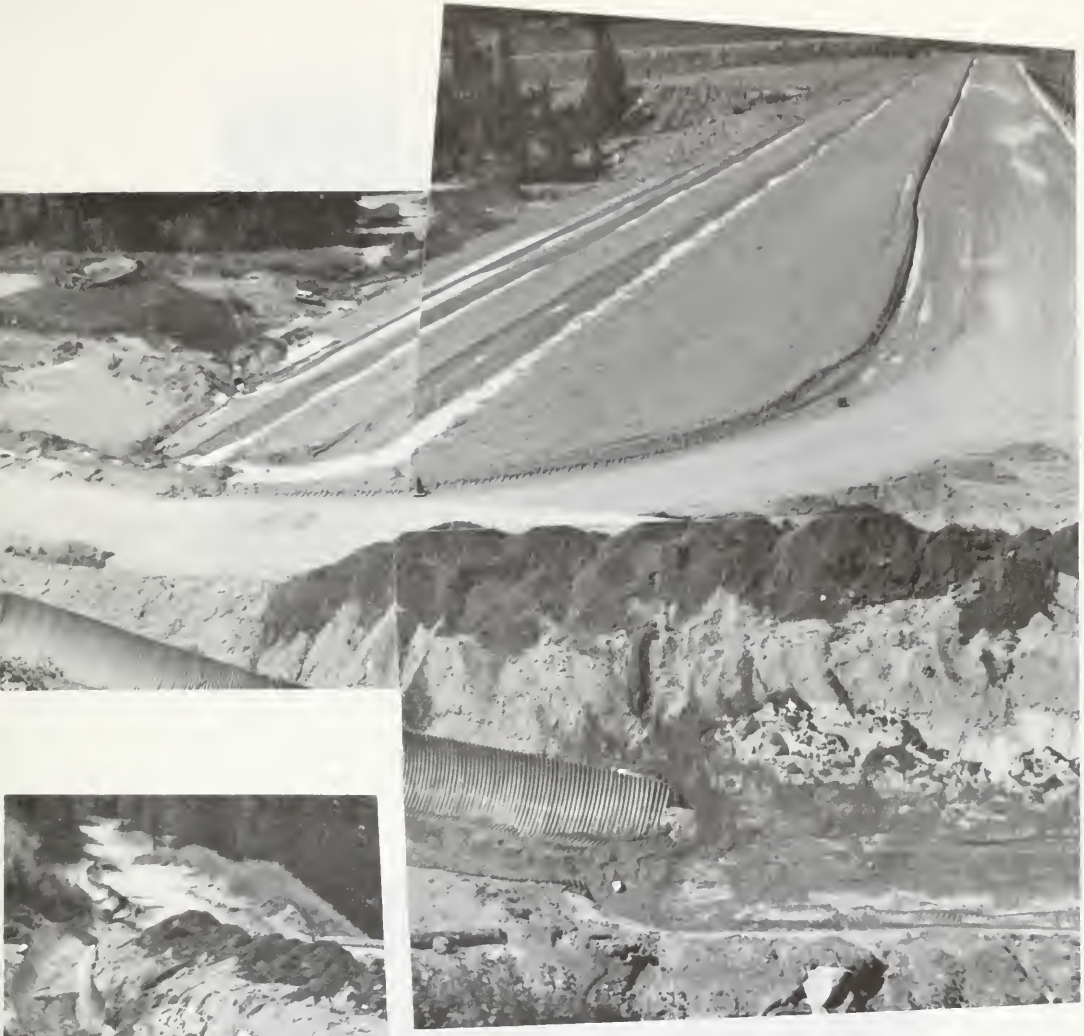


Photo 6 Zonolite Spillway and Downstream Face (8-13-81)

Photo 7 Spillway Chute and Approach Channel (8-13-81)

Photo 8 Spillway Approach Channel Looking Downstream (8-13-81)

Photo 9 Spillway Chute Looking Upstream Near Crest.
Note Overlapping Joints (7-81)

Photo 10 Spillway Foundation Collar (8-13-81)

Photo 11 Erosion of Backfill and Undermining of Spillway Chute (7-81)

Photo 12 Bends in Spillway Chute Looking Downstream (7-81)

Photo 13 Downstream End of Spillway Chute Adjacent to Haul Road (8-13-81)

Photo 14 East Abutment Drain for Benches on Downstream Face (8-13-81)

Photo 15 West Abutment Area. Note Rill Erosion (8-13-81)

Photo 16 Downstream Face Looking Towards the West (8-13-81)

Photo 17 Upstream Face Looking Towards the West (8-13-81)

Photo 18 Zonolite Tailings Dam - Phase 5 - Crest (8-13-81)

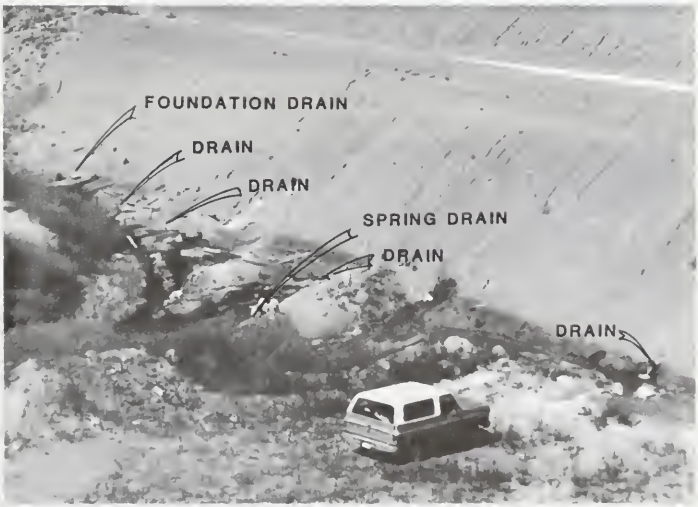


Photo 19 Foundation and Spring Drains. Note Wetted Front (8-13-81)

Photo 20 First Exposed Drain Closest to East (left) Abutment (8-13-81)

Photo 21 Seepage from Bedding Material Around Spring Drain (8-13-81)

Photo 22 Foundation and Spring Drains Looking Towards the East (8-13-81)

Photo 23 Third Exposed Foundation Drain from East Abutment (8-13-81)

Photo 24 Fourth Exposed Foundation Drain from East Abutment (8-13-81)

Photo 25 Fifth Exposed Foundation Drain from East Abutment (8-13-81)

APPENDIX

Correspondence

GRACE

Zonolite
Construction Products Division

W.R. Grace & Co.
P.O. Box 609
Libby, MT 59923

(406) 293-4131

Sept 4, 1981

Dept. of the Army
Seattle District, Corp. of Engineers
P. O. Box C-3755
Seattle, Washington 98124

Attn: R. P. Sellevold, P.E.
Chief, Engineering Division

Gentlemen:

W. R. GRACE & CO. would like to offer the following as comments to the Phase I inspection report - National Dam Safety Program - Rainy Creek Basin Zonolite Tailings Dam, Libby, Montana MT-1470 Dated July 1981 Revised.

Executive Summary Comments:

I. The location of the dam is stated to endanger lives in the event of a failure. Actually there is no residential area between the dam and the Kootenai River. Danger downstream of the mouth of Rainy Creek would be minimal because the Kootenai River, with a maximum flood stage rating of 95,000 CFS and with Libby Dam acting as flood control, is adequate to handle the flows projected by this report. A dam failure could force evacuation of the W. R. Grace facilities near the mouth of Rainy Creek, but to state that such an event would endanger lives is not justified in fact.

II. With only 30 days allowed to prepare comments, it is impossible for W. R. GRACE to have a hydrologic and climatologic study prepared. However, the probable maximum flood (PMF) presented in this report must be questioned. In order to judge the rational behind the PMF, we would like to present the following comparisons:

a. The PMF generates a 43,400 cfs flow in the 9.7 square mile basin. This is 35.9% of the maximum flood on record in the Kootenai River (121,000 CFS at Libby in 1916)

b. Average uncontrolled flow in the Kootenai River for 52 years on record is July 1, 30,000 cfs; Aug. 1, 13,000 cfs and is Sept. 1, 17,000 cfs.

c. If in fact this flow rate occurred and if the Zonolite Tailings Dam had spillway capacity to handle it (or if the dam did not exist and Rainy Creek was in it's natural channel), the amount of damage done thru the relatively steep canyon and at the confluence with the Kootenai, would make the integrity of the dam immaterial. Parts of lower Rainy Creek cascade down a 12% grade. Damage in the canyon, at the highway crossing and to the W. R. GRACE facilities near the mouth of the creek, would be extensive if the dam withstood the storm or not.

d. The volume predicted at 3770 acre - feet is 1.78 times the calculated volume retained behind the dam.

III. Spillway capacity is indicated to be 200 cfs. Using standard open channel flow calculation techniques, the capacity of the trapazoidal spillway feeding the 1/2 round lined channel is 760 cfs. The calculations show the 1/2 round discharge channel governs capacity.

IV. The stability analysis has been made available to C of E representatives. The question of the adequacy of the spillway cannot be addressed without correcting the calculated capacity of the existing spillway and without flow requirements from a rational PMF.

V. Comments on the Recommendations.

a. Any such storm as predicted in the report would provide adequate warnings to anyone down stream and no formal downstream warning plan is needed.

b. Any decant line leak which would present a problem would be detected by visual inspections and/or show up as an otherwise unexplained rise in the water level in the closest piezometer well.

c. The Zonolite tailings dam is not intended to provide flood storage or protection.

d. The question of adequate spillway capacity cannot be answered without reconsideration of design conditions.

e. W. R. GRACE has and intends to continue using competent engineers to monitor the condition of the dam.

W. R. GRACE & CO. appreciates the opportunity to comment on this report prior to publication. The magnitude of the probable maximum flood used in this report is questionable and the conclusions and recommendations based on it must be reconsidered based upon any changes.

Very truly yours,

CONSTRUCTION PRODUCTS DIVISION
W. R. GRACE & CO.

W. J. McCaig
General Manager
Libby Operations

WJM/ns

cc: J. Wolter, M. Ray

DEPARTMENT OF NATURAL RESOURCES
AND CONSERVATION
WATER RESOURCES DIVISION

TED SCHWINDEN GOVERNOR

32 SOUTH ... ING

STATE OF MONTANA

(406) 449-2872 ADMINISTRATOR
(406) 449-3962 WATER RIGHTS BUREAU
(406) 449-2872 WATER SCIENCES BUREAU
(406) 449-2864 ENGINEERING BUREAU
(406) 449-2872 WATER PLANNING BUREAU

HELENA, MONTANA 59620

September 10, 1981

Department of the Army
Seattle District, Corps of Engineers
P.O. Box C-3755
Seattle, Washington 98124

Attention: Ralph Morrison

Dear Ralph:

Re: Morrison-Maierle, Inc. Dam Safety Inspection Report of Zonolite Tailings Dam MT-1470.

We have reviewed the above referenced final draft report. We concur with the findings and recommendations and find that it satisfies the criteria of Phase I report.

Minor editorial comments have been discussed with your staff, and we understand these will be incorporated in the final report.

Thank you for this opportunity to review and comment on the final draft report on Zonolite Tailings Dam.

Sincerely,

Richard L. Bondy

Richard L. Bondy, P.E.
Chief, Engineering Bureau

RB:AT:lz

AN EQUAL OPPORTUNITY EMPLOYER

U. S. Department of Labor

Mine Safety and Health Administration
P O Box 25367
Denver, Colorado 80225

SAFETY AND HEALTH TECHNOLOGY CENTER
Mine Waste and Construction Division

September 1, 1981

Report No. D3674-W1499
File: HLS-5

MEMORANDUM FOR: WILLIAM C. GARDNER
District Manager, Rocky Mountain District
Metal and Nonmetal Mine Safety and Health

FROM: JOHN L. ODELL
Acting Chief, Mine Waste and Construction Division

SUBJECT: Review of Phase I Inspection Report, National Dam Safety Program by Morrison-Maierle, Inc. for the Zonolite Tailings Dam, near Libby, Lincoln County, Montana, Construction Products Division of W. R. Grace & Company

In response to a request by Wilbur Guthrie, Jr., Supervisory Mine Inspector, Salt Lake City Subdistrict Field Office, Helena, Montana, the Phase I Inspection Report, National Dam Safety Program, prepared by Morrison-Maierle, Inc., Consulting Engineers, was reviewed by the Safety and Health Technology Center, Denver, Colorado. The Seattle District Corps of Engineers has requested MSHA critique the report and submit their comments. The report was evaluated for compliance with MSHA's design guidelines for impounding structures associated with metal/nonmetal mines and MSHA's current regulatory standard, section 57.20-10, title 30, Code of Federal Regulations, which states, "if failure of a water or silt retaining dam will create a hazard, it shall be of substantial construction and inspected at regular intervals."

According to the report, the dam is located such that its failure could endanger many lives and cause excessive economic loss. The report also indicates that the hydraulic facilities can handle up to 45 percent of the probable maximum flood (PMF). Larger floods would therefore overtop the dam. Since failure can cause loss of life, this office recommends that the facilites be capable of handling 100 percent of the PMF.

The reservoir routing mentioned in the report was started at the minimum opening elevation of the decant tower. Since stoplogs will be placed in the decant opening as the tailings rise in the impoundment, it would appear prudent to begin the flood routing at the top elevation of the final stoplog.

Two pipes cross the spillway approach channel at an elevation only slightly above the channel invert. The pipes and any debris trapped by the pipes can significantly reduce flow through the approach channel. Each pipe should be relocated to eliminate any impediment of flow.

It appears that a portion of the approach channel sideslopes is formed by embankment material of the dam. These slopes should be properly lined to prevent erosion of embankment material.

The spillway chute, consisting of a half section of 8-foot diameter corrugated metal pipe and sloped at a relatively steep 8 percent, is only anchored at the dam crest. We concur with the findings stated in the report that the structural integrity of the spillway chute would be questionable when carrying design flows. Since a failure of the chute may affect embankment safety, revisions in design and construction should be made.

Although the report states that the computed safety factors, in regard to embankment stability, exceed minimum recommended allowable safety factors for both static and seismic loading, the safety factors obtained were not stated. Our guidelines require minimum static and seismic factors of safety of 1.5 and 1.2 respectively, under maximum normal anticipated phreatic conditions.

Plate 5 indicates that two piezometers are located on the upstream face of the phase 5 embankment construction. Although the piezometers would have been beneficial in locating the phreatic line during the initial phases of embankment construction, the piezometers during phase 5 will be too close to the water surface in the impoundment to provide any significant data.

It appears that the total stress method was used in determining shear strengths for the stability analysis. For long-term design, an effective stress analysis would have been more appropriate. Also, a dry density of 138 pounds per cubic foot and cohesion of 4000 pounds per square foot seems to be unusually high for gravelly sand material, especially the cohesion in a consolidated-undrained triaxial test. In essence, an embankment with a downstream slope of 2 horizontal to 1 vertical and constructed to the height indicated in the report can be very marginal in regard to obtaining a 1.5 static factor of safety. Therefore, this area of the investigation should be emphasized strongly to verify the designs.

The drawings and most of the report indicate the embankment is ultimately to be 125 feet high. It is assumed that the embankment will be limited to 125 feet rather than the planned 200 feet indicated on pages vii, 10, 11, and 29 of the report.

If we can be of further assistance, please let us know.

cc: T. Shepich
J. Mulhern
F. Delimba
W. Guthrie
R. P. Sellevold

DEPARTMENT OF THE ARMY
SEATTLE DISTRICT, CORPS OF ENGINEERS
P.O. BOX C-3755
SEATTLE, WASHINGTON 98124

NPSEN-FM

10 SEP 1981

Mr. William McCaig
W. R. Grace and Company
P.O. Box 609
Libby, Montana 59923

Dear Mr. McCaig:

Thank you for your comments regarding the Phase I inspection report on Zonolite Tailings Dam. While your comments will be considered in preparation of the final report, we will further address some of your noted concerns.

As the report indicates, the evaluation of the hazard potential is based on engineering judgement and is not supported by a detailed study and/or dam breach analysis. If, as you suggest, the hazard was down-graded from "high" to "significant," the inspection guidelines would still recommend the project (because of its size) be capable of handling the full PMF.

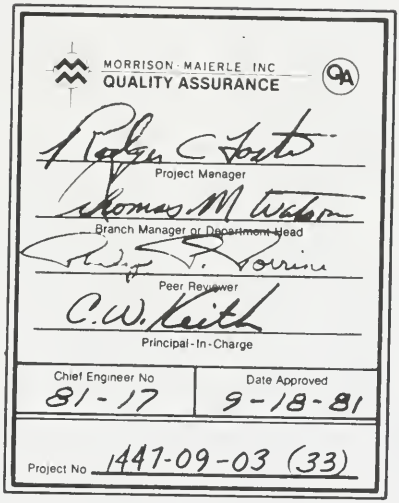
By definition, the PMF is the flood expected from the most severe combination of meteorologic and hydrologic conditions that are reasonably possible in a given region. The probable maximum precipitation for your area was obtained from data published by the U.S. Weather Bureau. We have recommended that more detailed studies be performed to more clearly define the downstream hazard and appropriate spillway design flood along with the recommended minimum water storage volume. The outflow capacity for the spillway is governed by the entrance to the half-round pipe spillway regardless of the approach channel capacity. The control section has a maximum flow capacity of 200 cfs. This outflow was used in the PMF routing.

Thank you for your assistance and cooperation in inspecting Zonolite Tailings Dam.

Sincerely,

K.P. Sellevold

R.P. SELLEVOLD, P.E.
Chief, Engineering Division


MORRISON-MAIERLE INC
QUALITY ASSURANCE
QA
Project Manager
Branch Manager or Department Head
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C.W. Keith
Principal-In-Charge
Chief Engineer No
81-17
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